A coastal resources division Report

An Assessment of Shore Erosion in Northern Chesapeake Bay

and of the Performance of Erosion Control Structures

1982

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We also thank Scott Zimmerman of the Maryland Natural Resources Police Force for piloting the airplane used to take aerial photos of the shoreline sites.

Preparation of this report was funded in part by NOAA, Office of Coastal Zone Management, and by the Maryland Department of Natural Resources.

EDITORS' NOTE

For over fifty years, erosion-control structures have been built in the northern Chesapeake Bay, either alone or in networks that stretch continuously along the shoreline. A recent study by the U.S. Army Corps of Engineers, The Chesapeake Bay Future Conditions Report (1977), finds that some areas with structural protection have persistent erosion problems, and other areas have aged or failing structures which are in need of improvement. The State of Maryland funds a program in the Department of Natural Resources with the purpose of providing the financial aid and engineering expertise that is needed to build erosion-control structures in problem areas.

Decisions on maintaining the existing network of erosion-control structures, and on the public funding of new erosion-control projects in the northern Bay need to be made with some understanding of the performance of existing structures, and of the coastal processes and shoreline characteristics for which new structures need to be designed.

This document describes selected shoreline structures in Maryland's portion of the Chesapeake Bay, and discusses the physical processes of coastal erosion which affect their performance. The information that has been collected as part of this study was used to answer some important questions about shore erosion:

1. What are examples of success and failure of erosion-control structures?

2. What geologic and hydrologic factors affect erosion and the performance of structures along the different types of Bay shoreline in Maryland?

3. Do different types of land use cause different amounts of shore erosion?

This study report contains engineering evaluations of forty cases of structures which protect shoreline sites ranging from high bluffs to low banks, beaches, and marshes. The types of structures include: bulkheads, groins, revetments, gabions, and well-rings. Each structure is presented to illustrate its effectiveness in controlling fastland loss, and specific recommendations are provided to improve the siting and design of similar structures in future shoreline situations. The report also discusses the coastal processes along the main Bay shoreline, and illustrates their relationship to the historic rates of coastal retreat in different shorefront areas. Finally, the report describes land-use patterns along substantial portions of the northern Chesapeake Bay in Maryland, and discusses how the changing patterns of land use in shorefront areas can be related to historic erosion rates.

The data and analysis contained in this report provide answers to the questions above which should be useful to engineers, managers, decision-makers, and other persons who participate with interest in public forums and related discussions where the protection of the Chesapeake Bay shoreline against further erosion is regarded as a significant management issue.

Chris Zabawa
Chris Ostrom
September 1, 1982

EXECUTIVE SUMMARY

This report describes a study undertaken by the Maryland Department of Natural Resources to evaluate different types of erosion-control structures, as well as several environmental factors which control rates of shore erosion, such as waves, tides, storms, and littoral sediment transport. A portion of the report contains forty "case studies" of shore erosion-control structures built around Maryland's Chesapeake Bay shoreline. Each "case study" included a site visit by a coastal engineering consultant to make important observations on the condition and performance of the structure in its shoreline environment. Another portion of this report describes the coastal processes that are responsible for erosion, and a statistical analysis which examined all the factors for their relationship to the historic erosion rate around the Bay margins.

The major result of the study was that well-designed and constructed erosion-control structures are effective in stopping local shoreline erosion in the northern Chesapeake Bay, regardless of local geology, coastal morphology, wave energy, or other environmental parameters. Structures are successful in the northern Bay partially because of the relatively mild wave climate compared to the open ocean coastline. Several relatively low cost ($150.00 per foot) designs for structures can be effective along the northern Bay shoreline, but the individual characteristics of each structure (such as seawall elevation or revetment stone size) must depend on the particular physical setting.

The results of the field evaluations by the independent engineering consultant showed most of the structures were successfully controlling erosion, even at the sites which had historic erosion rates exceeding ten feet per year before the structures were built. In a few cases the consultant suggested an alternative design which might perform better at a particular shoreline site. These suggestions are included in this report, along with the consultant's observations on the condition and performance of the structures, and with photographs which illustrate the effectiveness of each structure in controlling the shoreline loss. Where available, information was also included in each "case study" about the initial cost of the structure and about the preconstruction engineering cross-section.

Five of the forty "case studies" showed substantial deterioration which was judged to have been preventable through a different design, more effective maintenance program, or better understanding of the coastal processes.

The deficiencies which were noted most of the time during the site visits were:

- overtopping of structures by waves
- lack of periodic maintenance and repair of damage to structures from storms or winter ice.

After reviewing the modes of failure of some of the structures, the consultants recommended sloping revetments ("rip-rap") as their preferred strategy for many more shoreline situations on Maryland's Bay. This is because the materials used (stone) do not degrade with time; this type of strategy is less likely to fail catastrophically during a storm; there is less scour of sediment on the seaward sides of these structures; and the "rip-rap" generally provides a better habitat for biota than the treated wood or concrete used in other types of shore protection.

Besides recommending sloping revetments for wider application on Maryland's Bay shorelines, the consultants also recommended using filter material in erosion-control structures under all circumstances, and more frequent maintenance of many erosion control structures.

In designing and maintaining structures, serious consideration needs to be given to the combination of maximum tides and waves (run-up) which can be expected in the lifetimes of structures on the northern Chesapeake Bay. The height of structures necessary to prevent overtopping by waves depends on both the normal water depth at any shoreline site, and the maximum potential wave height. All structures as a minimum, should be designed for top elevations greater than the "annual" storm run-up to avoid serious damage due to wave overtopping. A simple procedure for determining the adequate heights of structures anywhere on Maryland's Bay shoreline is presented in Chapter III.

The initial results of the statistical analysis seem to indicate that modelling the pattern of historic erosion rates around the edges of the main Chesapeake Bay in Maryland cannot be suitably done by using traditional regression of discriminant analysis procedures. Areas with low, medium, or high rates of coastal retreat were found to possess many similar characteristics of wave energy, tide, littoral sediment movement or other factors. But, there were no characteristics (such as high levels of wave energy, or high levels of littoral sediment movement) which were found to be unique to areas of high erosion rates.

CHAPTER I

INTRODUCTION

Hsiang Wang, Robert Dean,
Robert Dalrymple, Robert Biggs, and Randall K. Spoeri

A. Purpose of the Study

The shorelines of Chesapeake Bay are experiencing an erosion trend averaging approximately 2-3 feet per year. Since 1968, the State of Maryland through the Department of Natural Resources has maintained a program for technical and financial assistance to Bay-front property owners to mitigate erosion and, as of 1979, a total of $6.8 million in public funds had been appropriated for this purpose.

The magnitude of this program and the importance of proper shoreline management are such that there is considerable interest in ensuring that the best designs for erosion control structures are developed. This study evaluates the present design basis for existing structures and contains design recommendations for future erosion control structures which are based on environmental information that was synthesized from many sources to describe the wave, current, and wind forces acting on the northern Chesapeake Bay shoreline.

B. Contents of this Report

Chapter II contains descriptions of forty case studies of shore erosion structures along the northern Chesapeake Bay shoreline which were selected for evaluation to provide a variety of types and shoreline conditions. Each case study presents a brief description of the type of materials and installation at the particular shoreline site, and assesses the performance of each structure relative to the wave and storm conditions which can be expected. For the structures which had major flaws in design or construction, a corrective method is presented. Otherwise, the structures are rated on their present condition and comments are provided which are intended to help improve the performance of the different structural types when they are installed at new shoreline sites.

Chapters III and IV summarize the results of the field observations of the forty "case studies", and develop recommendations for a future erosion control strategy in the northern Chesapeake Bay.

Chapter V contains descriptions of the geologic and hydrologic characteristics along the Chesapeake Bay shoreline in Maryland, developed from field data and computer models. Each factor (shoreline type, tide range, storm surge, littoral drift, wave energy, and rainfall) is analyzed in a qualitative manner for its relationship to the pattern of historic erosion rates around the edges of the Chesapeake Bay in Maryland.

Chapter VI describes an objective statistical analysis of the shoreline, which was undertaken with the data on individual reaches developed in Chapter V, in an effort to mathematically model the historic erosion patterns. The historic erosion rate was studied as a function of five explanatory variables:

- dominant shoreline type
- mean tide range
- "100-year" storm surge
- wave energy
- littoral drift.

Three statistical methodologies were employed:

- descriptive statistical analysis
- regression analysis
- discriminant analysis

The statistical analyses which were performed to mathematically model the erosion rates used computer program packages and the actual reach data.

Finally, Chapter VII examines the changes in land-use patterns which have occurred over the last few decades along substantial portions of the northern Chesapeake Bay shoreline, and discusses the relation of these different land uses to the patterns of shore erosion on the same shoreline reaches.

C. Results

The major result of the study was that well-designed and constructed erosion control structures are effective for stopping local shoreline recession in the northern Chesapeake Bay, regardless of the coastal morphology, wave energy, or other environmental parameters. Many of the forty "case studies" were judged to be satisfactorily controlling the shoreline loss in areas where the historic rate of coastal retreat had been as high as ten feet per year, or more before the structures were installed. Structures are successful in the northern Chesapeake Bay partially because of the relatively mild wave climate compared to the open ocean coastline.

Another result of the study was that there is no strong relationship between any single cause of erosion and the historic pattern of coastal retreat around the edges of the northern Chesapeake Bay. The initial results of the statistical analysis seem to indicate that modelling the pattern of historic erosion rates around the edges of the main Chesapeake Bay in Maryland cannot be suitably done by using traditional regression of discriminant analysis procedures. The analysis is necessarily preliminary in nature, and further statistical tests may provide more conclusive results.

There are many known characteristics which influence shoreline erosion rates:

- waves, currents, and storm conditions,

- type of material being eroded,
- presence of vegetation along the shore,
- height of bluff being eroded,
- sheltering provided by offshore islands or offshore bars
- length of shoreline,
- runoff or rainfall, seepage from bluff faces,
- the freeze/thaw cycle,
- effects of nearby shoreline structures,
- sea level rise.

At any one shoreline location the erosion rate may be due to a combination of the above factors. Areas with low, medium, or high rates of coastal retreat were found to possess many similar characteristics of wave energy, tide, littoral sediment movement or other factors; but, there were no characteristics (such as high levels of wave energy, or high levels of littoral sediment movement) which were found to be unique to areas of high erosion rates.

After reviewing the modes of failure of some of the structures in the northern Chesapeake Bay, Coastal and Offshore Engineering and Research Inc. (COER) recommended sloping revetments as their preferred strategy for many more shoreline situations. This method for erosion control offers the following advantages:

(1) The materials used to build revetments do not degrade with time.

(2) Sloping revetments are unlikely to fail catastrophically. (If design conditions should be exceeded slightly during a storm, inevitably some stones may become dislodged and can be replaced afterwards.)

(3) Wave reflection from sloping revetments is usually low; thus, less disturbance and less scour of sediments results at the toe of the structure.

(4) Rubble generally provides a better habitat for biota than the materials which are used in most other types of shore protection.

Another recommendation is for serious consideration to be given to the combination of maximum tides and waves (run-up) which can be expected in the lifetimes of structures on the northern Chesapeake Bay shoreline. Chapter III explains a simple method for determining the proper wall height of revetments, and the proper weight of the stone armor suggested for structures at any shoreline site. These recommendations will result in the cost of revetments exceeding the cost of timber bulkheads by approximately 30%, (timber bulkheads, as presently designed in the northern Chesapeake Bay, cost approximately 30% more than stone revetments as presently designed.)

D. Conclusions

Based on the forty case studies of erosion control structures, it is concluded that:

(1) Relatively low cost (= $150/ft) erosion control structures can be effective in controlling shore erosion around the northern Chesapeake Bay.

(2) Revetments need to be strongly considered for future erosion situations due to their inherent durability.

(3) Filter material needs to be used in erosion control structures under all circumstances.

(4) Field demonstrations are needed to assess the effectiveness of innovative structures. Two examples are: (a) a groin compartment filled with sand selected to be sufficiently coarse that it will not move offshore during storms, and (b) the possible use of gabions only as a top stabilizing layer with the interior formed of smaller angular rock fragments.

(5) Most structures in the northern Chesapeake Bay do not cause substantial beach erosion alongshore. In part, this is because erosion of the fastland does not introduce large amounts of suitably-sized sand into the beach system. Thus, there are not substantial quantities of sand in littoral drift which can be intercepted by groins with subsequent sand starvation on adjacent "down-drift" beaches.

(6) Erosion control structures need to be monitored following extreme events, such as hurricanes or severe winter storms. The information obtained will aid in future selection and design of structures.

(7) The procedure developed in Chapter III for establishing the crest elevations of protective vertical walls should be systematically incorporated into the design of new structures.

(8) There needs to be frequent monitoring and maintenance of coastal structures. Protective coatings need to be maintained on hardware, sheeting, and pile tops. Splits in wood need to be mended on aging bulkheads, and any backfill which has washed out behind vertical protective structures needs to be replaced. Flanking erosion at the ends of any structure needs to be stopped. Dislodged stones in revetments need to be repositioned.

(9) Under some circumstances, man's activities such as land use could have an effect on shore erosion, but an attempt to quantify the effect of land use on erosion rates was not successful because it is not possible to locate an area where the land use had remained stable over a time span of several years for which there is a known erosion rate. However, based on coastal engineering and geological considerations different types of land use (agriculture, woodlands, urban) are not expected to be a dominant factor influencing erosion.

(10) The existing data on rainfall distribution were also compiled as part of this study to be used to assess the effect on erosion of bluff shorelines in different areas, but the existing data were found to be insufficient for this purpose.

CHAPTER II

EVALUATION OF EROSION CONTROL STRUCTURES

Robert Dean, Hsiang Wang,

Robert Biggs, and Robert Dalrymple

A. Introduction

Many different kinds of engineering structures have been installed along the northern Chesapeake Bay shoreline (Figure 2.1) to protect sites ranging from high bluffs to low banks, beaches, and marshes. The shoreline sediments which are armored by the structures range from gravelly sands to stiff clays. The structures themselves must endure a wide range of different wave, tide, longshore current, and storm conditions. This chapter presents "case studies" of forty different sites where structures have been installed for erosion protection. Each structure is presented to illustrate its effectiveness in controlling fastland loss, and COER, Inc., was asked to make specific recommendations which would improve the siting and design of similar structures in future shoreline situations.

The forty case studies include:

- 14 sites with timber bulkheads
- 4 sites with concrete bulkheads
- 3 sites with aluminum bulkheads
- 1 site with asbestos cement bulkhead
- 16 sites with stone revetments
- 4 sites with gabions
- 15 sites with groins

Next Pages: Figure 2.1. Schematic drawings showing types of erosion control structures installed along northern Chesapeake Bay shoreline.

Figure 2.1

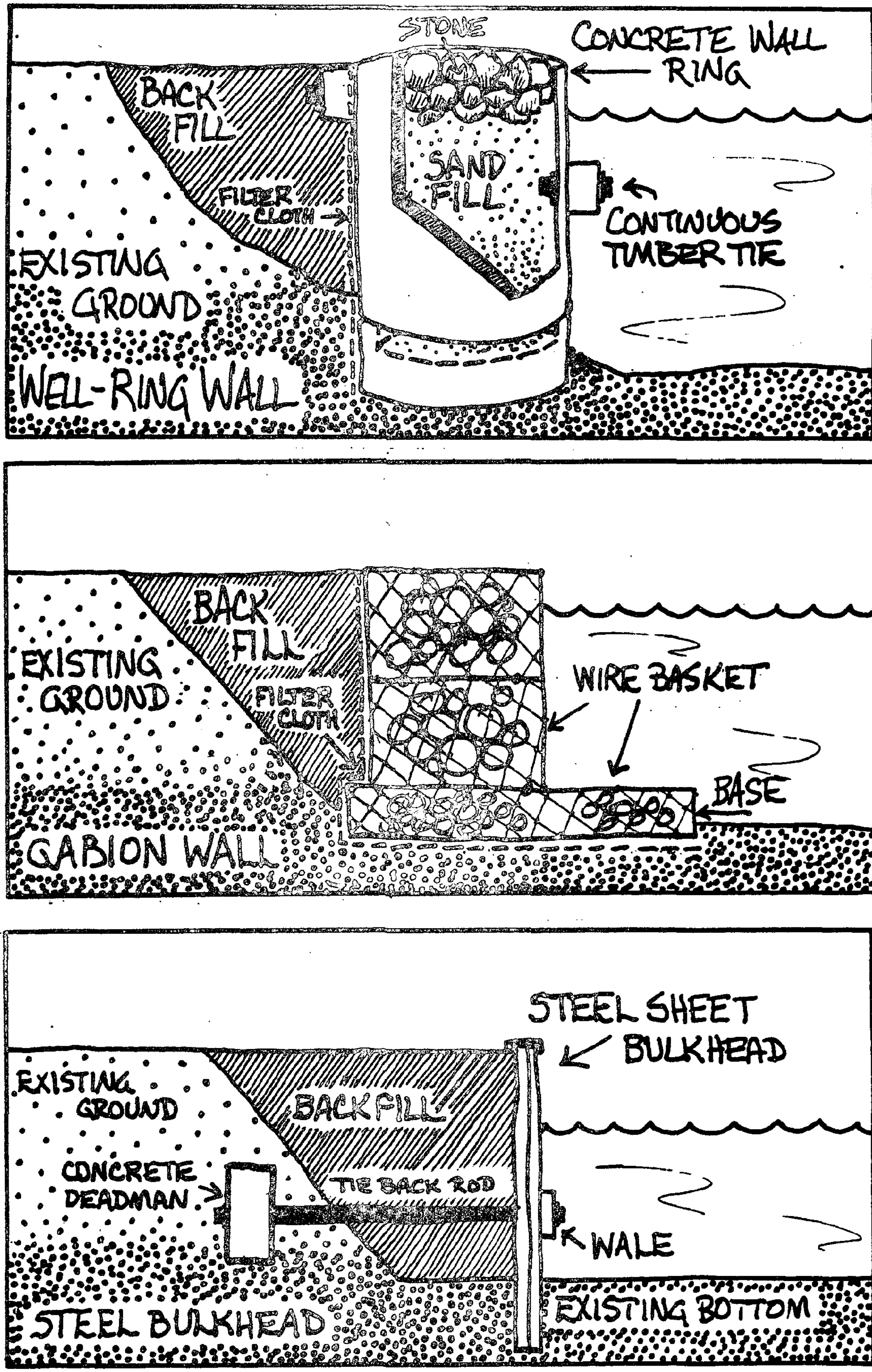
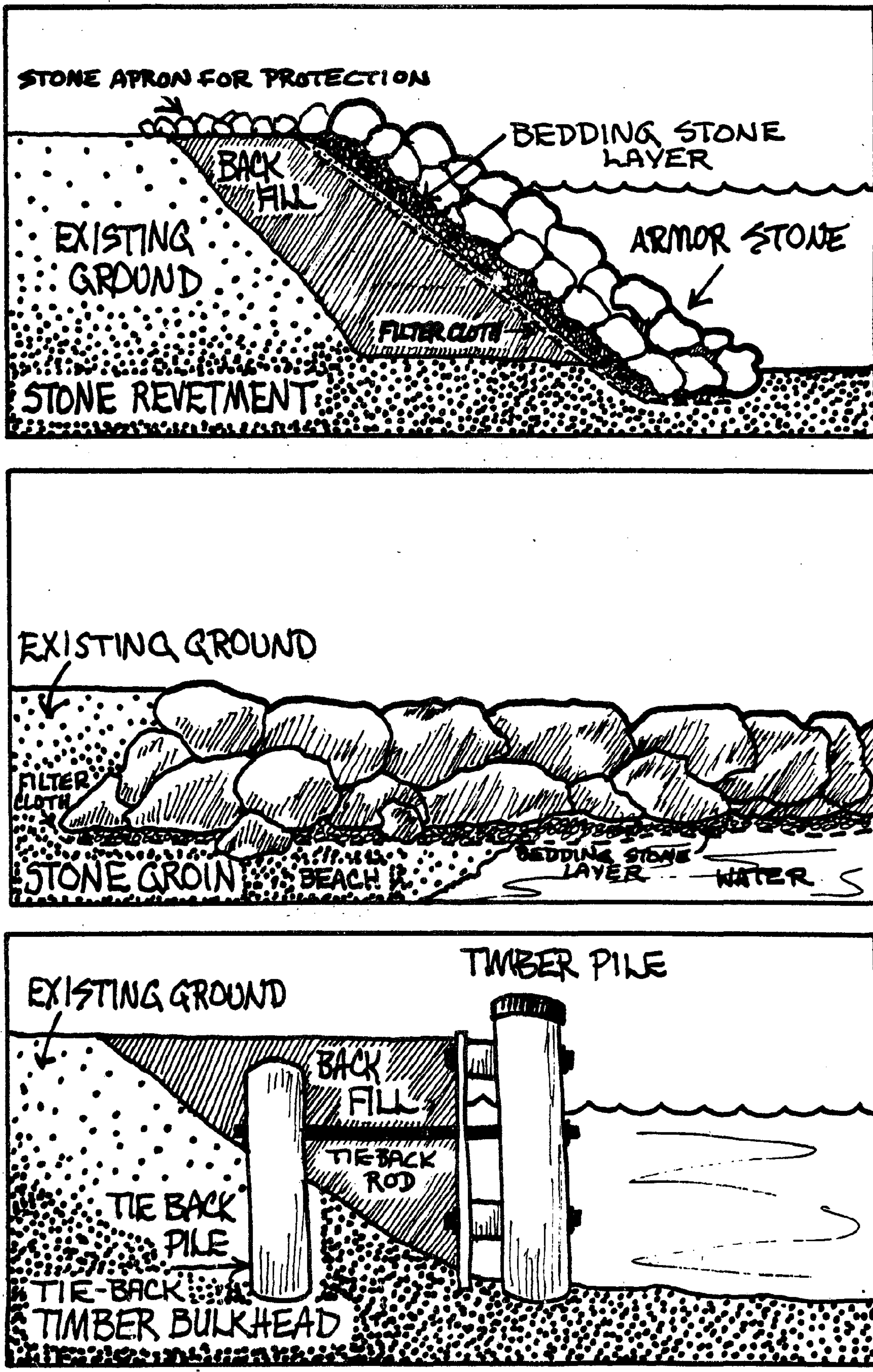

STONE
CONCRETE WALL RING
BACK FILL
SAND FILL
FILTER CLOTH
CONTINUOUS TIMBER TIE
EXISTING GROUND
WELL-RING WALL
BACK FILL
EXISTING GROUND
FILTER CLOTH
WIRE BASKET
BASE
GABION WALL
STEEL SHEET BULKHEAD
EXISTING GROUND
BACK FILL
CONCRETE DEADMAN
TIE BACK ROD
WALE
EXISTING BOTTOM
STEEL BULKHEAD

Figure 2.1


STONE APRON FOR PROTECTION
BEDDING STONE LAYER
BACK FILL
EXISTING GROUND
ARMOR STONE
FILTER CLOTH
STONE REVETMENT
EXISTING GROUND
FILTER CLOTH
BEDDING STONE LAYER
STONE GROIN
BEACH
WATER
TIMBER PILE
EXISTING GROUND
BACK FILL
TIE BACK ROD
TIE BACK PILE
TIE-BACK
TIMBER BULKHEAD

- 1 site with well-rings
- 1 site with concrete pipes

The following pages discuss the evaluations on a case-by-case basis, for different portions of the Chesapeake Bay shoreline in Maryland. Each "case study" contains photographs along with descriptions of engineering criteria, hydrologic conditions, and nearshore characteristics. Where available, information is also included about the cost of the structure at the date of construction, about the historic erosion rate at the site before the structure was built, and about the pre-construction engineering cross-section.

The results of the field evaluations show most of the structures are successfully controlling coastal retreat, even at the study sites which had historical erosion rates exceeding ten feet per year or more before the structures were installed. In a few cases, COER, Inc. suggested an alternative design which might perform better at a particular shoreline site. Five of the forty case studies showed substantial deterioration which was judged to have been preventable through a different design, more effective maintenance program, or better understanding of the coastal processes.

This investigation of a number of successful and unsuccessful shore erosion-control structures shows the need for property owners to use competent engineering expertise and top-of-the-line construction methods when planning and building a new shore protection project. There is a real potential for resources to be wasted through a combination of quick designs and improper construction methods. This report is not intended to address the economic considerations involved in projects requiring the least expenditure, and therefore, a smaller and more compact structure. Rather, the "case studies" were selected to illustrate the performance of structures along different types of shoreline on the Chesapeake Bay in Maryland, and to show instances of success and failure of structures.

It is important to note in the case studies that construction costs per linear foot are stated for the year of expenditure. These costs can be adjusted to 1980 levels through a published method of cost indexing contained in the weekly engineering magazine Engineering News Record. The method for determining approximate costs for construction is contained on the following page in Table 2.1. This method may not indicate the true inflation rate for marine construction, but is included for illustration purposes only.

The costs of structures are also reflected in actual 1980 bid prices for shore erosion-control projects built by the Department of Natural Resources Shore Erosion Control Program.

Type of Structure	Number of Structures	Average Cost Per Foot	Range of Costs Per Foot
stone revetments	18	\$124.24	\$101.00 - \$219.18
aluminum bulkhead	2	\$141.19	\$135.47 - \$199.89
timber bulkhead	15	\$211.54	\$161.33 - \$328.47

$$\frac{\text{Total Cost}}{\text{Total Footage}} \quad \frac{\$2,213,183.60}{15,383.4} = \$143.87 \text{ average cost per foot.}$$

Table 2.1
from: Engineering News Record

BUILDING COST INDEX HISTORY 1913-1980

How ENR builds the Index: 68.38 hours of skilled labor at a 20-cities average of bricklayers', carpenters' and structural ironworkers' rates, plus 25 cwt of standard structural steel shapes at the mill price, plus 22.56 cwt (1.128 tons) of Portland cement at a 20-cities average price, plus 1,038 feet of 2 x 4 lumber at a 20-cities average price.

BUILDING COST INDEXES:

1913 = 100

1954 = 446	1963 = 594	1972 = 1048
1955 = 469	1964 = 612	1973 = 1138
1956 = 491	1965 = 627	1974 = 1204
1957 = 509	1966 = 650	1975 = 1306
1958 = 525	1967 = 672	1976 = 1425
1959 = 548	1968 = 721	1977 = 1545
1960 = 559	1969 = 790	1978 = 1674
1961 = 568	1970 = 836	1979 = 1819
1962 = 580	1971 = 948	1980 = 1943

EXAMPLE: TO COMPUTE A CONSTRUCTION COST INCREASE FROM 1974 TO 1980:

(a) Divide 1980 index by 1974 index:

$$1943 \div 1204 = 1.61$$

(b) Multiply to adjust 1974 cost to 1980 level:

1974 cost x 1.61 = 1980 cost

Adjusted costs determined by this method will be below changes in the CPI (Consumer Price Index) as published by the U.S. Department of Labor, and may not indicate the true inflation rate for marine construction. This is included for illustration purposes only.

B. Cases along the Lower Eastern Shore of the Delmarva Peninsula

This area of the northern Chesapeake Bay contains portions of the shoreline in Dorchester County, Wicomico County, and Somerset County (Figure 2.2). The sections below present a brief physical description of the shorelines and coastal processes, followed by a discussion of the case studies which were selected from this area.

SHORELINE DESCRIPTION

Dorchester County The Chesapeake Bay shoreline in Dorchester County runs from the mouth of the Choptank River to Hooper Island. Shorefront areas contain heavily-wooded lands, agricultural fields, and some scattered residential development with shoreline structures at different points.

The shoreline along most of Trippe and Brannock Bays and Taylors Island is composed of exposed eroding banks which generally range from 3-6 feet high. The width of the beach at the base of these banks is extremely variable. At some sites, a beach is absent in front of the banks and trees growing along the land's edge are falling off the banks into the water. In other areas, a beach is present which extends landward into wooded areas, or onto farmlands. This is evidence of active beach erosion and coastal retreat.

Next Pages: Figure 2.2. Shoreline along the lower eastern shore of the Delmarva Peninsula in Maryland.

Figure 2.3. Some representative shoreline profiles collected in the summer of 1980 along the lower eastern shore of the Delmarva Peninsula in Maryland.

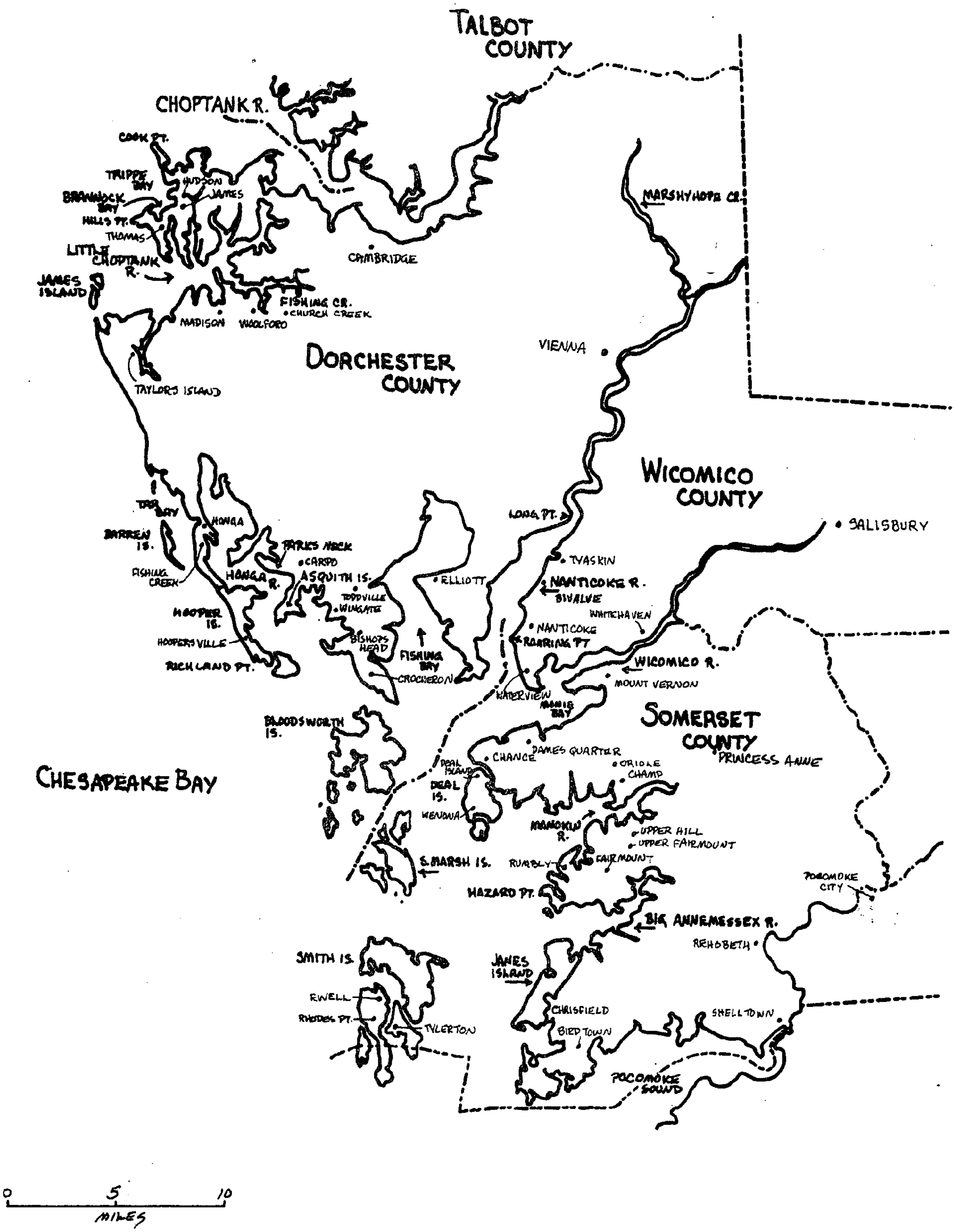
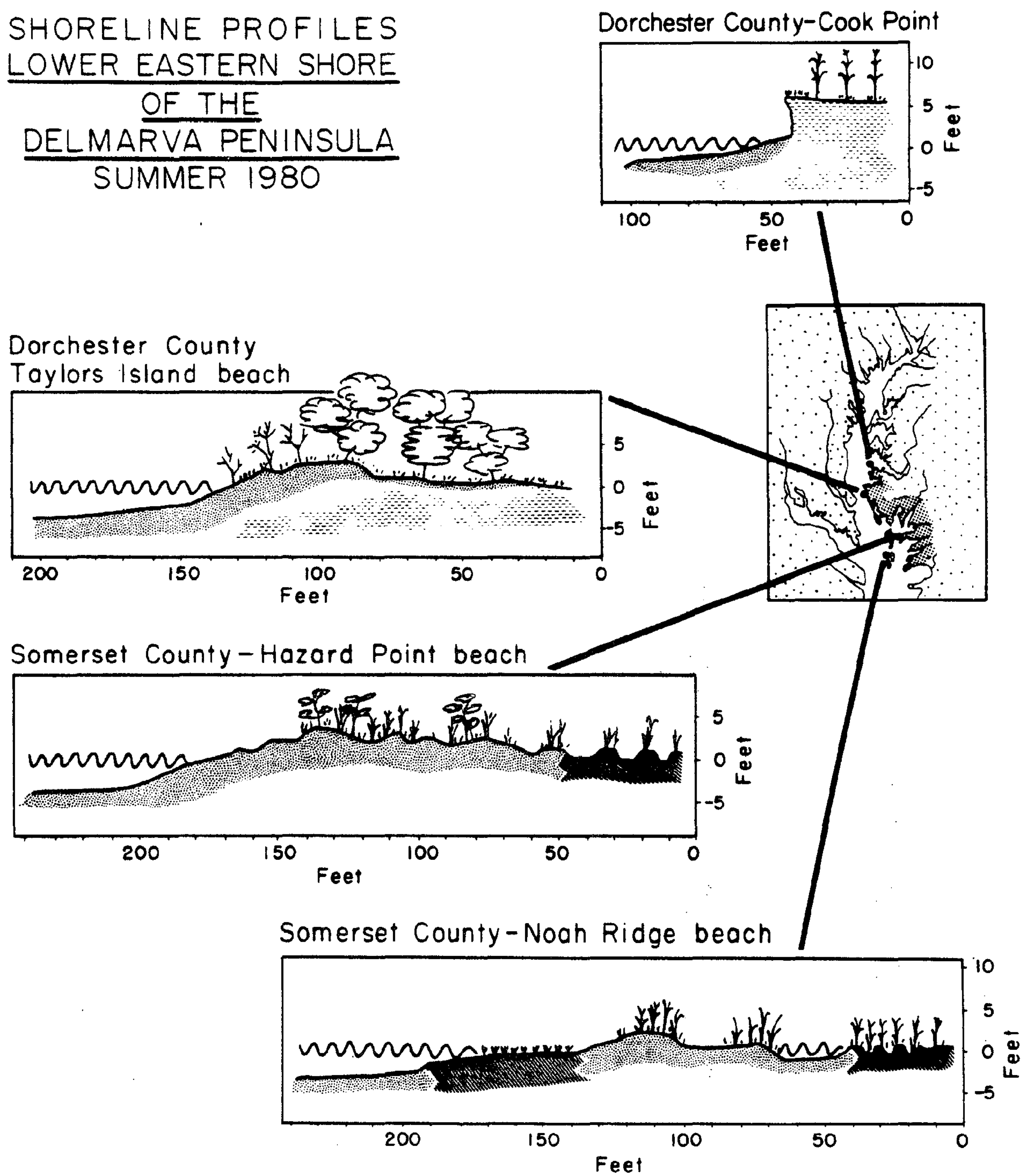

TALBOT COUNTY
CHOPTANK R.
COOK PT.
TRIPPE BAY
HUDSON
JAMES
BRANNOCK BAY
HILLS PT.
THOMAS
LITTLE CHOPTANK R.
JAMES ISLAND
CAMBRIDGE
FISHING CR.
CHURCH CREEK
MADISON
WOOLFORD
MARSHYHOPE CR.
VIENNA
DORCHESTER COUNTY
TAYLORS ISLAND
WICOMICO COUNTY
SALISBURY
LONG PT.
TYASKIN
NANTICOKE R.
BIVALVE
WHITEHAVEN
NANTICOKE
ROARING PT.
WICOMICO R.
MOUNT VERNON
TAR BAY
BARREN IS.
HONGA
FISHING CREEK
HONGA R.
HOOPER IS.
HOOPERSVILLE
RICHLAND PT.
ASQUITH IS.
ELLIOTT
TODDVILLE
WINGATE
BISHOPS HEAD
FISHING BAY
CROCHERON
WATERVIEW
MONIE BAY
SOMERSET COUNTY
PRINCESS ANNE
BLOODSWORTH IS.
CHESAPEAKE BAY
CHANCE
DAMES QUARTER
ORIOLE
CHAMP
DEAL ISLAND
DEAL IS.
WENONA
MANOKIN R.
UPPER HILL
UPPER FAIRMOUNT
S. MARSH IS.
RUMBLY
FAIRMOUNT
HAZARD PT.
POCOMOKE CITY
BIG ANNEMESSEX R.
REHOBETH
SMITH IS.
JANES ISLAND
EWELL
RHODES PT.
TYLERTON
CRISFIELD
BIRD TOWN
SHELLTOWN
POCOMOKE SOUND
0
5
10
MILES

Figure 2.3


SHORELINE PROFILES
LOWER EASTERN SHORE
OF THE
DELMARVA PENINSULA
SUMMER 1980
Dorchester County-Cook Point
10
5
0
-5
Feet
100
50
0
Feet
Dorchester County
Taylors Island beach
5
0
-5
Feet
200
150
100
50
0
Feet
Somerset County-Hazard Point beach
5
0
-5
Feet
200
150
100
50
0
Feet
Somerset County-Noah Ridge beach
10
5
0
-5
Feet
200
150
100
50
0
Feet

On Hooper Island, the shoreline contains many erosion structures protecting residential development. The lower portion of Hooper Island is largely undeveloped, and marshes along the shoreline are interrupted by beaches of varying lengths. Some of these beaches contain berms or small vegetated dunes.

Behind Hooper Island, the shoreline on the Honga River and in Fishing Bay is composed principally of marsh which ends abruptly at the water's edge in most spots. Some small pocket beaches are present for distances of 200-2000 feet along the shore. A few of these beaches stretch for longer distances and are backed by isolated banks between 3 and 8 feet high. Shorefront development in this lower portion of Dorchester County is largely restricted to the areas shown on the map.

Wicomico County A small part of the Wicomico County shoreline which was included in the study runs along the lower reaches of the Nanticoke River estuary below Long Point. Between Long Point and the Town of Nanticoke, the shoreline is composed of vegetated banks fronted either by sandy beaches or marsh. In some areas, the beach profile extends landward into wooded shorefront areas, and trees next to the beach are dying or falling into the water. There is enough sand present at most points to form berms on the shoreline profiles, and the beach sediments are stabilized by shrubs and beach grasses. Shorefront homes have been built all along this reach, both on low sandy flats next to the beach, or on higher ground which slopes gently down to the water's edge. At the Towns of Bivalve, Tyaskin, and Nanticoke, many structures have been installed to protect shorefront development.

A long arcuate beach extends along the shore south from the Town of Nanticoke to Roaring Point. This beach protects a heavily wooded area, and enough sand is present in the shoreline system at this point to form wide vegetated berms and small dunes in front of the trees. A large sand spit extends out from the land at Roaring Point almost to the main channel of the Nanticoke River.

Below Roaring Point, the shoreline is composed largely of beaches backed by sandy banks ranging from 3 to 10 feet. Most shorefront homes are separated from the water by a buffer strip of lawn grass, beach and berms. Where woodlands are present next to the shore, the beaches may be interrupted by stands of trees or small marshes which extend down to the water's edge.

Somerset County Much of the Somerset County shoreline is composed of marsh with pocket beaches extending from 500-2000 feet along the shore. In the Big Annemessex River and Manokin River estuaries, marsh sediments form the northern shores, and long arcuate beaches form the southern shores. Small vegetated dunes are observed landward of these beaches in some areas. The remaining shorefront areas on Janes Island, on Cedar Island, and along Pocomoke Sound are also composed of marsh and intermittent beaches of variable length which often have small vegetated dunes.

The Smith Island shoreline contains marsh interlaced with many tidal creeks. Along Tangier Sound, the Smith Island shoreline contains marsh which ends abruptly at the water's edge in most spots. On the Chesapeake Bay side of Smith Island, the shoreline contains some small sandy beaches and dunes. In many spots, these sand deposits are located only between marsh and the waterline. Immediately seaward, the underlying marsh sediments are once again exposed in the nearshore zone of breaking waves.

The shoreline on Deal Island and Dames Quarter is markedly different than in the lower portions of Somerset County. Here, most of the shoreline contains exposed sandy banks at least 6 feet high with beaches of varying widths at the waterline. There is also a large remnant sand dune in the middle portion of Deal Island. Coastal residential development extends along much of the shoreline of Deal Island and Dames Quarter together with many groins and bulkheads.

Shorefront development and bulkheading are also present on the Somerset County shoreline at the towns of Ewell and Tylerton on Smith Island, around Crisfield, and at several smaller towns along the Big Annemessex and Manokin Rivers.

Coastal Processes Many of the historical erosion rates for the lower Eastern Shore are greater than 8 feet/year (see Chapter V). The shoreline sediments which are eroded include the fine-grained sands of the Quarternary and Kent Island Formations, and younger marsh and alluvial deposits (Appendix A).

The mean tide range varies in the area from 1.3 to 2.1 feet, depending on the shoreline location. The storm surges from "annual" storms are between 2-3 feet at all locations, and the surges from "100-year" storms can be greater than 4 feet above mean low water. Waves during these severe storms can be as high as 4 feet on top of the storm surges.

Waves in the area approach from the northwest and southwest with the longest fetches. But the waves travel across large shoals offshore in many spots, and the wave energy can dissipate somewhat before reaching the beach. Thus, wave conditions on windy days and during "annual" storms are not exceptionally severe.

The wave and storm conditions are discussed in greater detail along with the other coastal processes in Chapter V.

Case Studies The structure case studies selected in this area include:

	Case No.	Structure
•	1	A stone revetment on Taylors Island (598 feet long).
•	2	A stone revetment on Taylors Island (206 feet long).
•	3	A stone revetment on Taylors Island (990 feet long) with 2 stone groins each 60 feet long.
•	4	A stone revetment in front of a timber bulkhead on upper Hooper Island (95 feet long).
•	5	A stone revetment (295 feet long) with 2 stone groins (104 feet total length) in Tar Bay.
•	6	A stone revetment on upper Honga Island (686 feet long).
•	7	An aluminum bulkhead at Parks Neck on the Honga River (502 feet long).

Case No.	Structure
• 8	A timber bulkhead on Asquith Island in the Honga River (827.5 feet long).
• 9	A timber bulkhead in Trippe Bay (727 feet long).
• 10	A timber bulkhead (266 feet long) on the south shore of the Choptank River with 2 stone groins, each approx. 50 feet long.
• 11	A segmented stone revetment, in Brannock Bay (727 feet long).
• 12	A stone revetment (1069 feet long) on the south shore of the Choptank River.

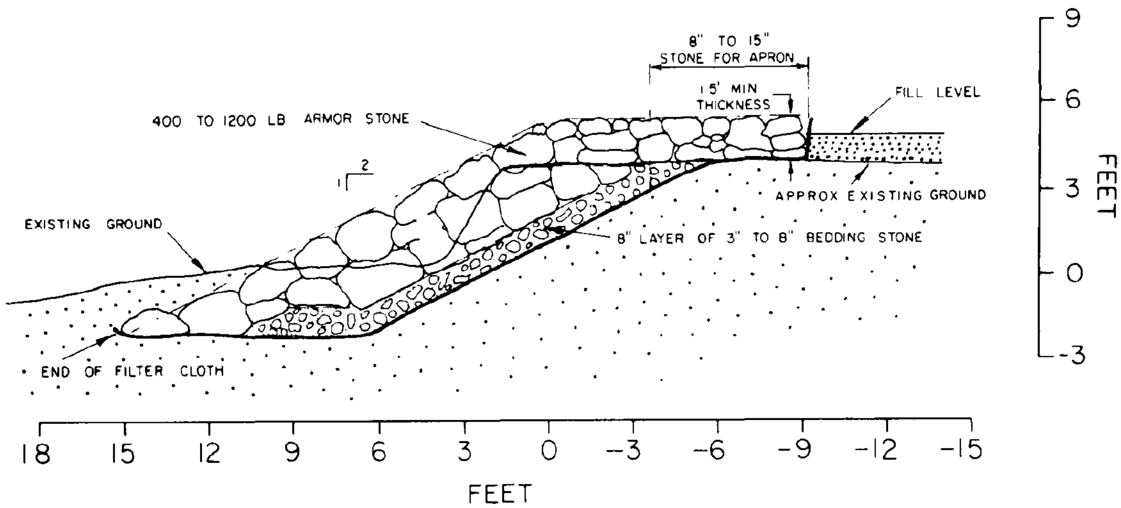
The following pages present brief descriptions of each structure, and near-shore bottom profiles collected at the sites. Most of the cases assigned for the lower Eastern Shore were in generally fair condition. Some structures were flanked by erosion at points alongshore, and almost all of the structures showed evidence of wave splashover. A few of the structures do not have adequate height to prevent wave overtopping during severe storms as shown on the cross sections on the following pages.

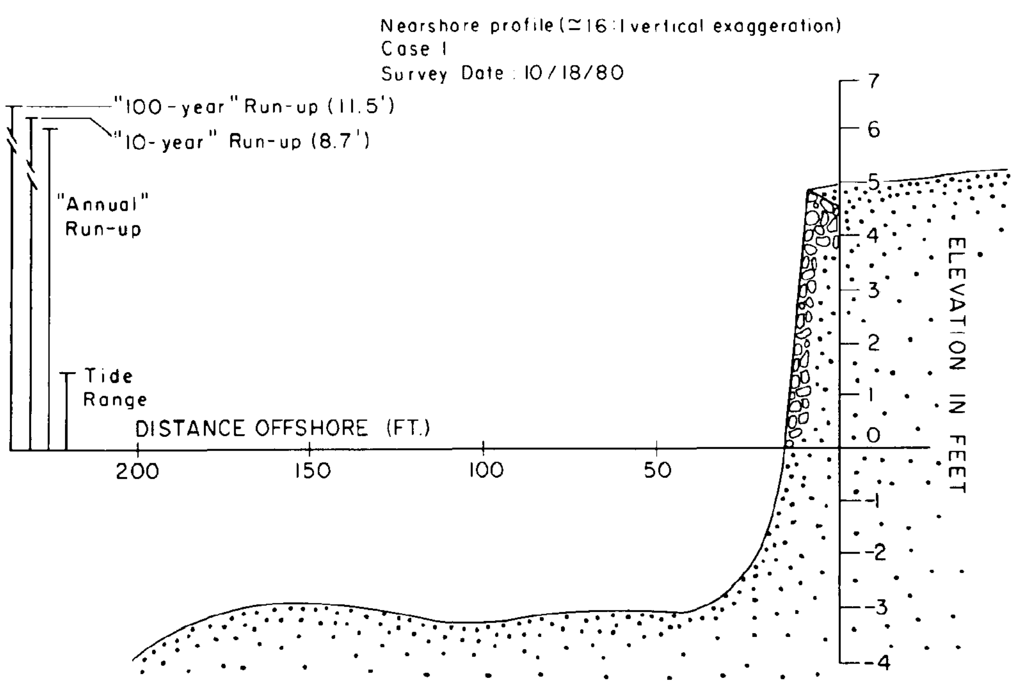
For Case No. 7, a redesign is suggested here to correct a serious deficiency due to the short return walls or the failure to extend these walls sufficiently landward as erosion continued. The historical erosion rate is about 5 feet/year and the return walls with their 60° angles have an effective landward extent of only 12.6 feet. Theoretically this short wall at a site whose historical erosion rate is 5 feet/year requires remedial efforts at least every 2.5 years, since in this amount of time, the flanking erosion reaches the end of the wall. Over 5 years have elapsed since construction with no additional wall extension, and a considerable amount of fastland loss has occurred. An appropriate design would be 90° angled flanked walls (i.e. perpendicular to the beach front), thus providing the greatest landward distance. These walls should be over 25 ft. in length, thus providing a minimum of 5 years flanking protection. At the end of 3 to 5 years, an additional wall extension would have to be driven. Periodic extensions of the flank wall would have to be made in the future as well as on an as-needed basis.

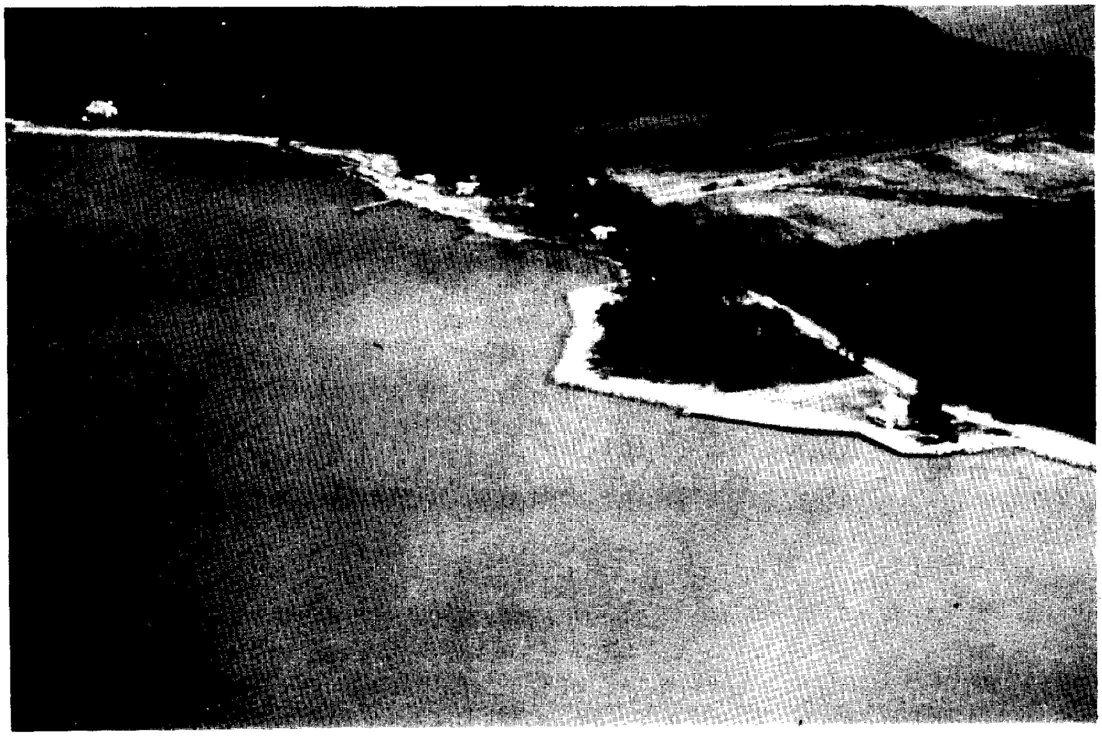
More discussion on the design of return walls to prevent flanking of new structures is contained in Chapter III. A reliable method for selecting the proper wall elevation for revetments and vertical walls is also described in Chapter III.

CASE 1 A STONE REVETMENT ON TAYLORS ISLAND

Structure was completed in 1978 at a cost of $84.56/ft. The historical rate of erosion at the site was 10.5 ft./yr. from 1848-1942. Stone revetment, on a 2:1 slope, consists of 400-1200 lbs. stone in a 3 ft.-thick armor layer. A bedding layer 8 in. thick was placed below the armor layer. Bedding layer consists of 3-8 in. stone. Filter material was used below bedding layer. This structure is in generally good condition. Severe erosion and failed structures can be observed directly adjacent to this case.



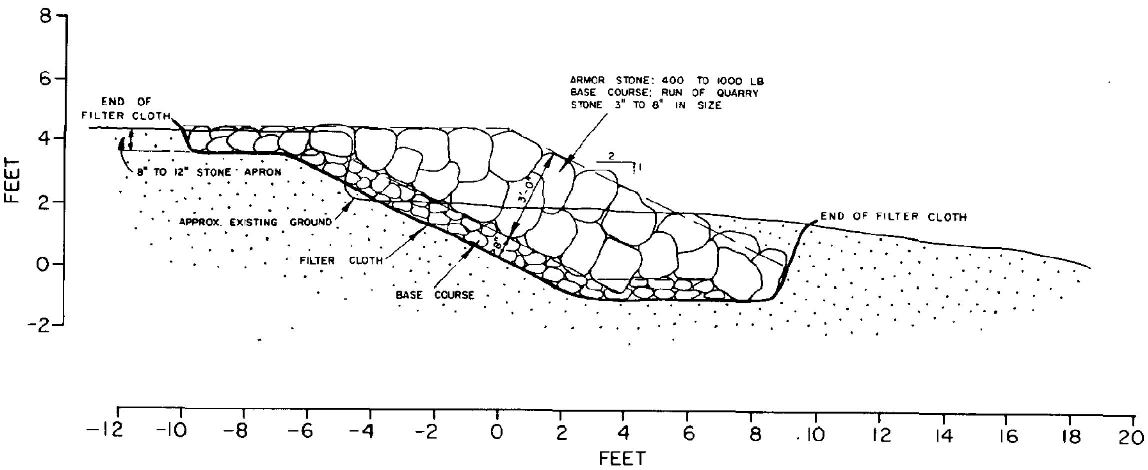


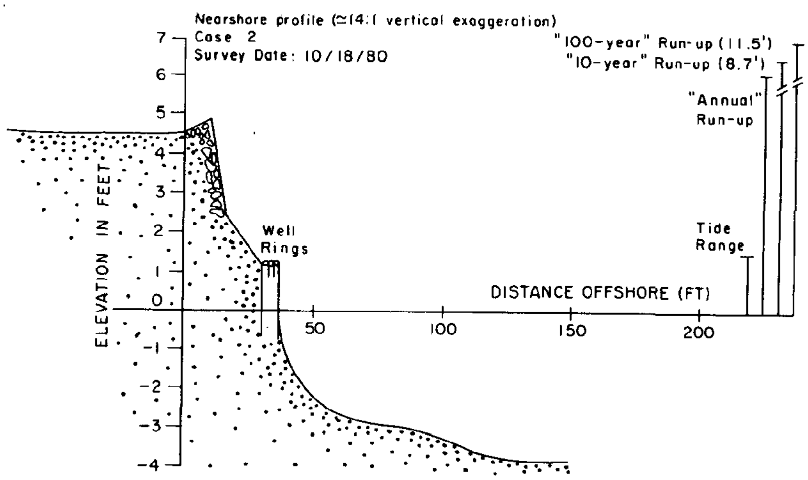


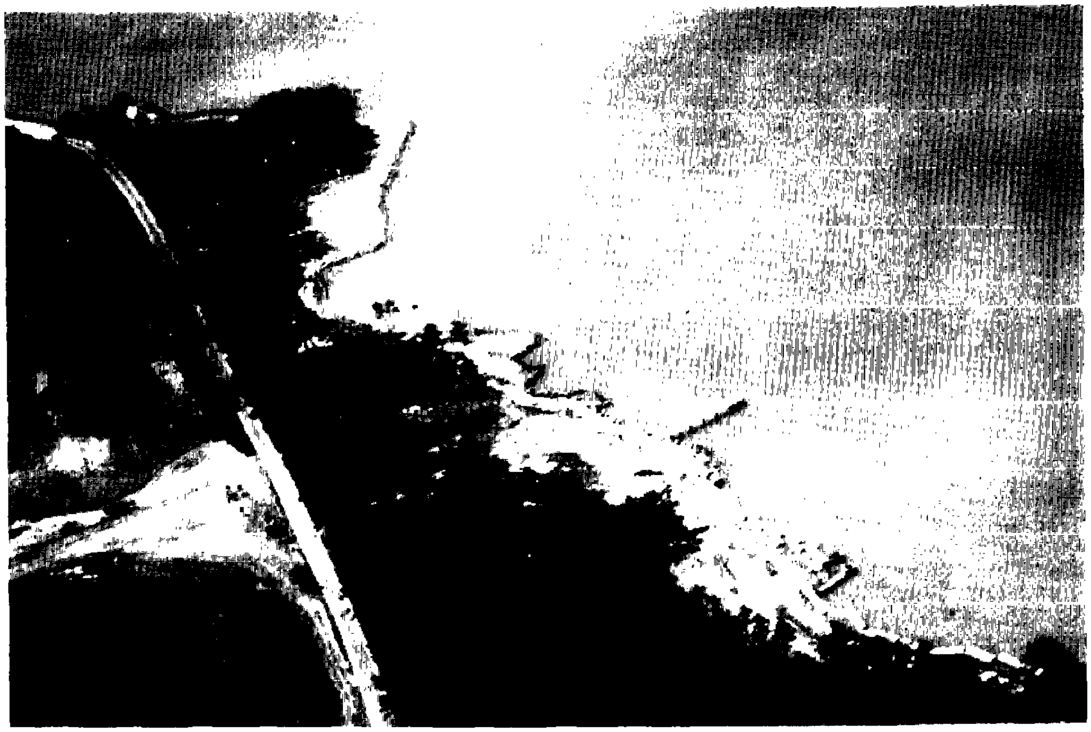
CASE 1 A STONE REVETMENT ON TAYLORS ISLAND

CASE 2   A STONE REVETMENT ON TAYLORS ISLAND

Structure was completed in 1977 at a cost of $72.82/ft. The historical rate of erosion at the site was 10.5 ft./yr. from 1848-1942. Stone revetment, on a 2:1 slope, consists of 400-1000 lbs. stone in a 3 ft.-thick armor layer, with 8 in.-thick bedding layer. Revetment has m-shaped well-ring structure on seaward side, and connects to a well-ring structure alongshore in one direction. There is flanking erosion alongshore in the other direction. Sand has not been impounded in the lobes of the well-ring wall. This structure is in generally fair to good condition, but is constructed of smaller stone than other revetments along the same shoreline reach.



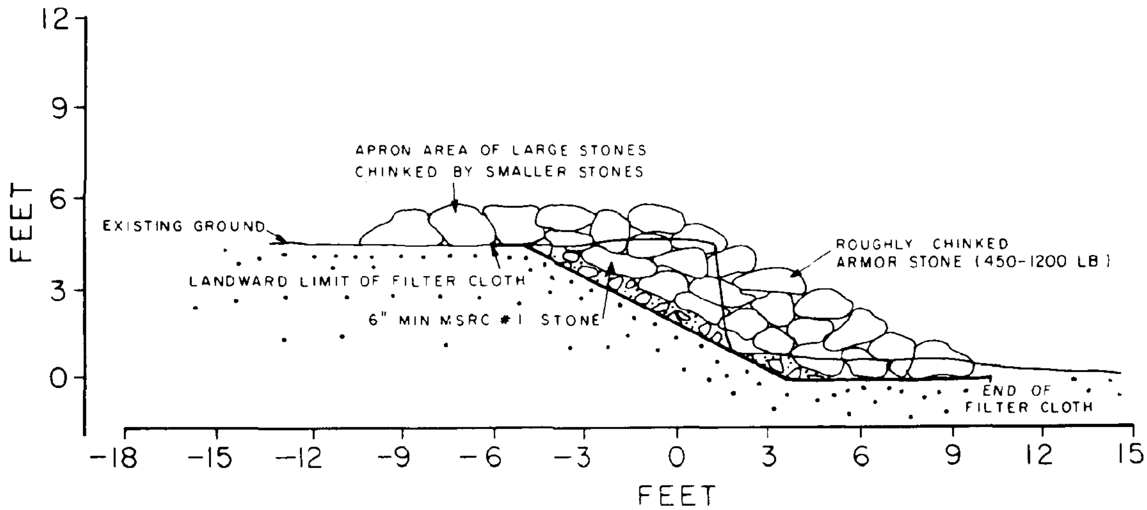


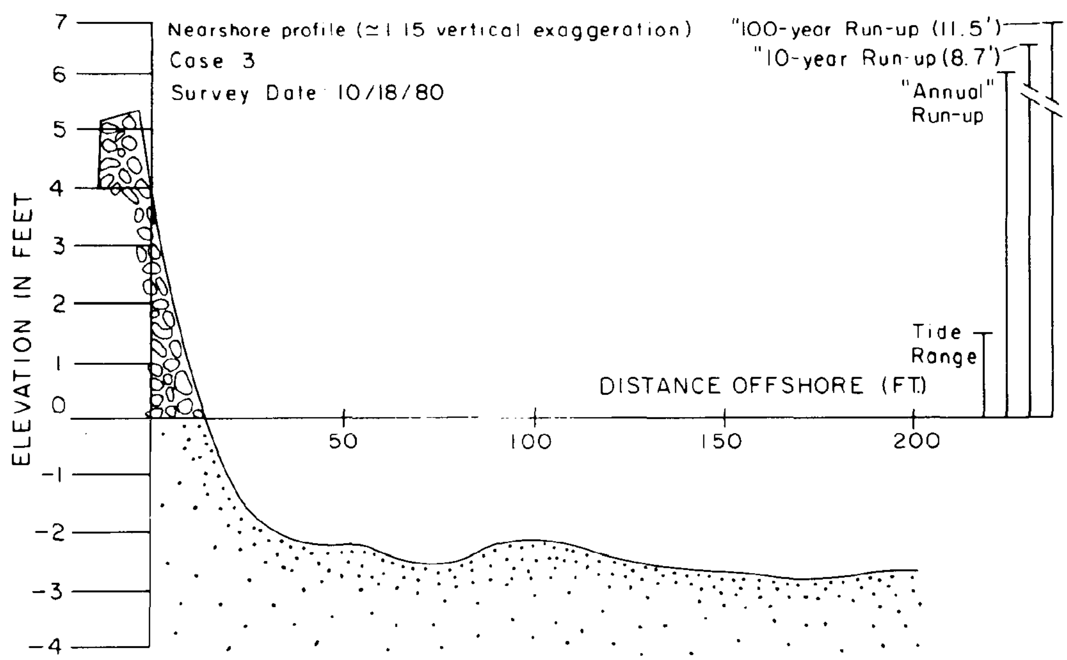


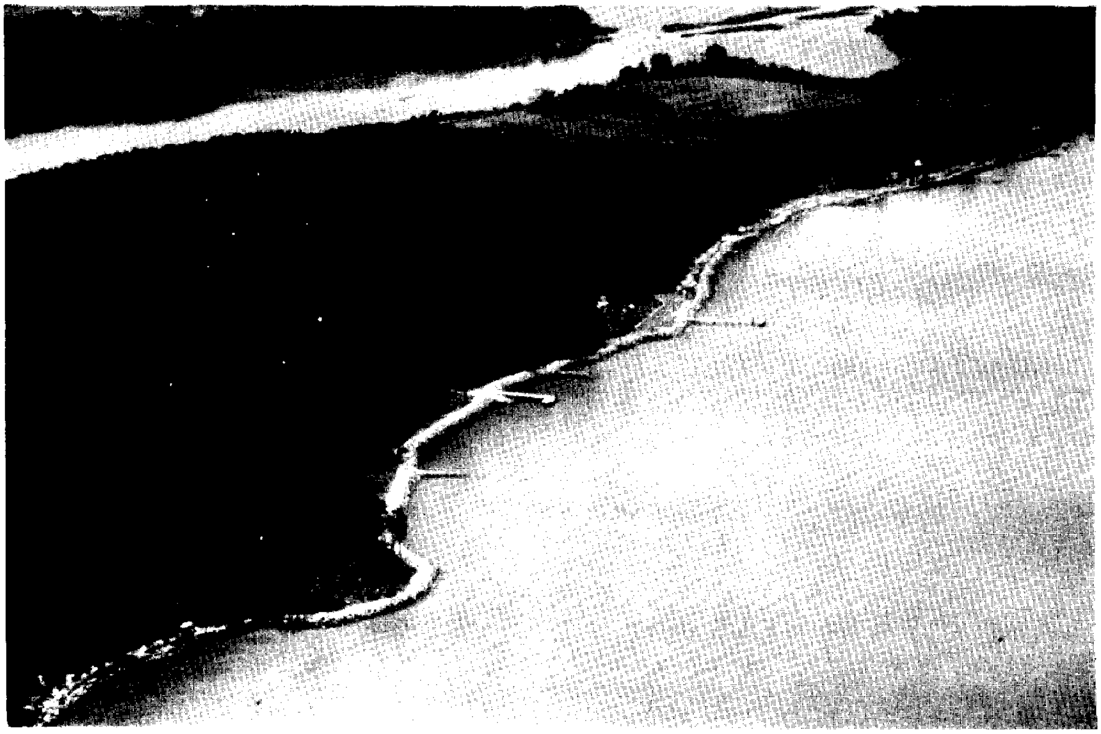
CASE 2 A STONE REVETMENT ON TAYLORS ISLAND

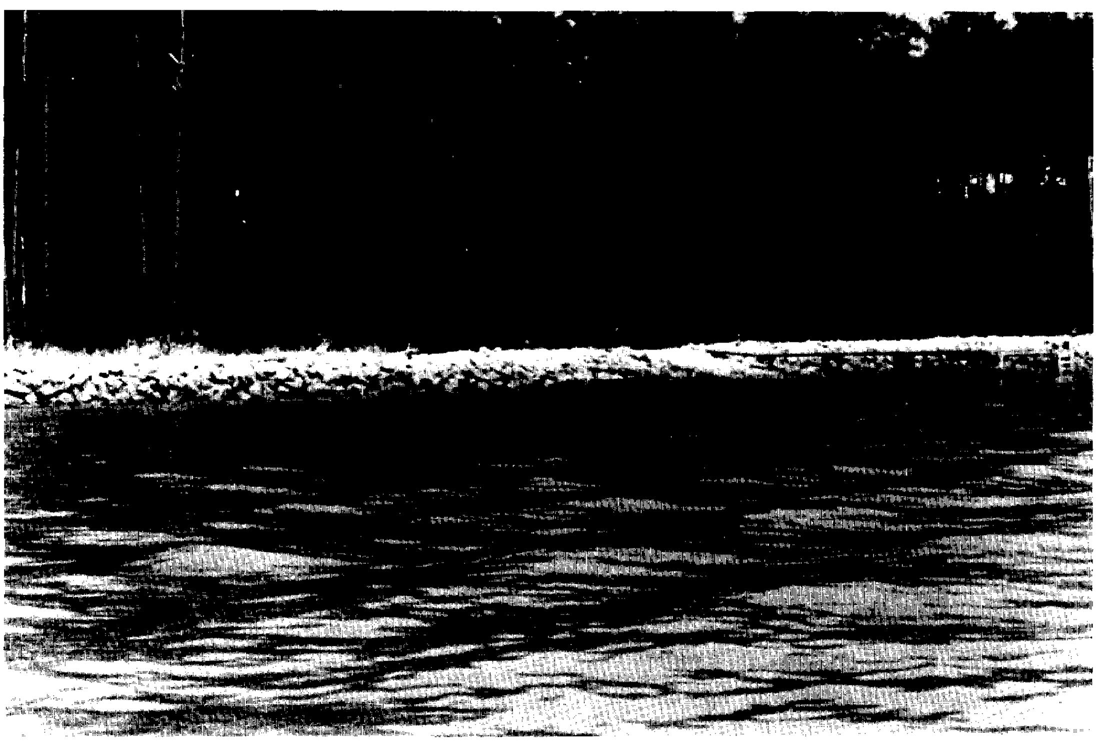
CASE 3 A STONE REVETMENT WITH STONE GROINS ON TAYLORS ISLAND

Structures were completed in 1975 at a cost of $44.80/ft. for the revetment, and $50.00/ft. for the groins. The historical rate of erosion at the site was 10.5 ft./yr. from 1848-1942. Stone revetment, on a 2:1 slope, consists of 450-1200 lbs. stone in a 2.5 ft.-thick armor layer. A bedding layer 6 in. thick, composed of small stone, was placed below the armor layer. Filter material was used below bedding layer. Additional splash apron, 13 ft. wide, was installed. Two stone groins appear to be serving no useful purpose, as no beach was observed in the summer of 1980. This structure is in generally good condition. Owners report the revetment sustained no damage during Tropical Storm David (Sept. 1979). The presence of submerged tree stumps offshore is evidence of high erosion rates prior to installation of structure.





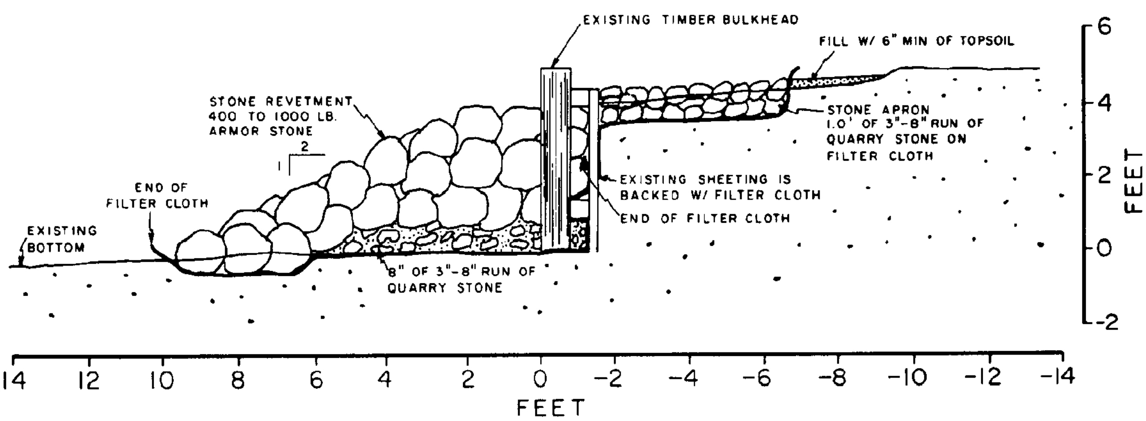


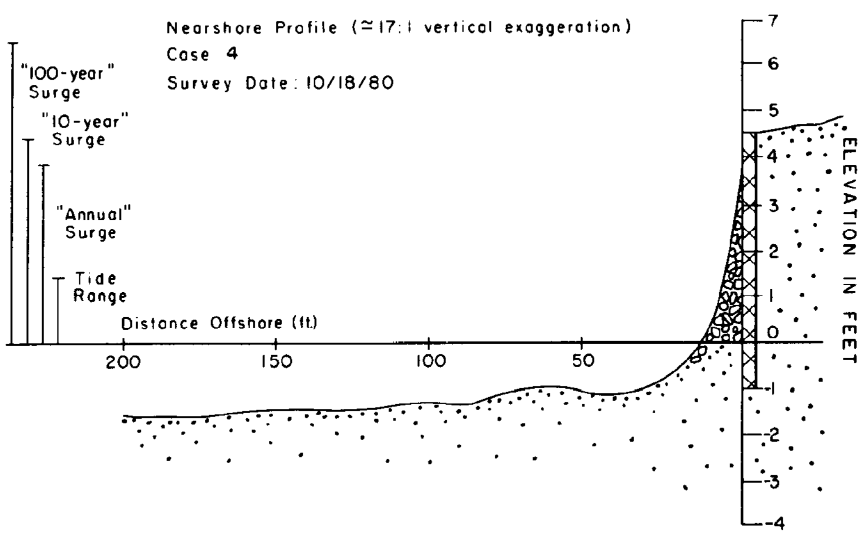


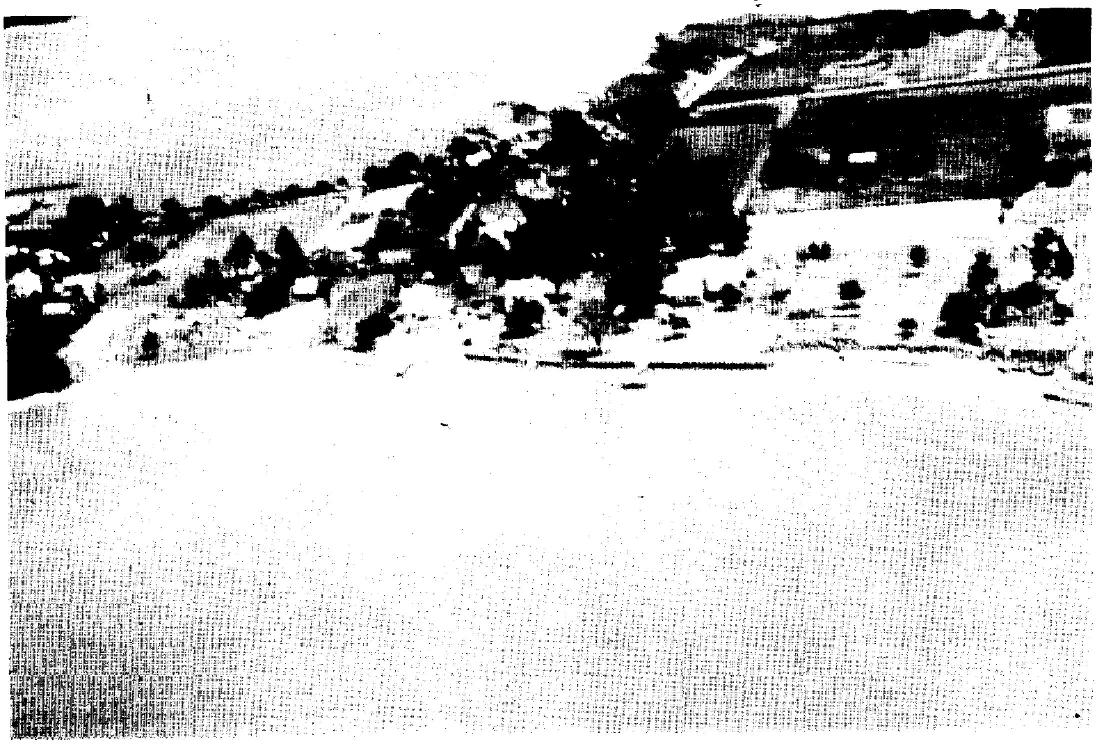
CASE 3 A STONE REVETMENT WITH STONE GROINS ON TAYLORS ISLAND

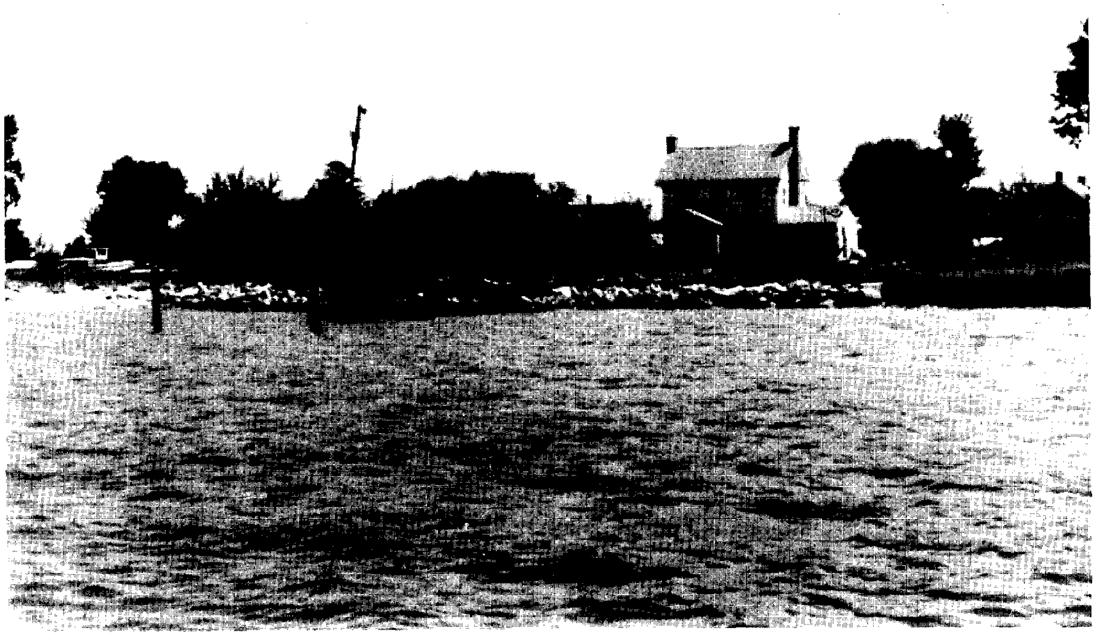
CASE 4 A STONE REVETMENT IN FRONT OF A TIMBER BULKHEAD ON UPPER HOOPER ISLAND

Structure was completed in 1977 at a cost of $67.55/ft. The historical rate of erosion at the site was 1-2 ft./yr. from 1848-1942. Stone revetment, on a 2:1 slope, consists of 400-1000 lbs. stone. Revetment fronts a timber bulkhead. Splashover apron, composed of 3-8 in. stone, is installed behind the timber bulkhead. Filter cloth was used under both the revetment and the stone apron. This structure is in generally good condition.









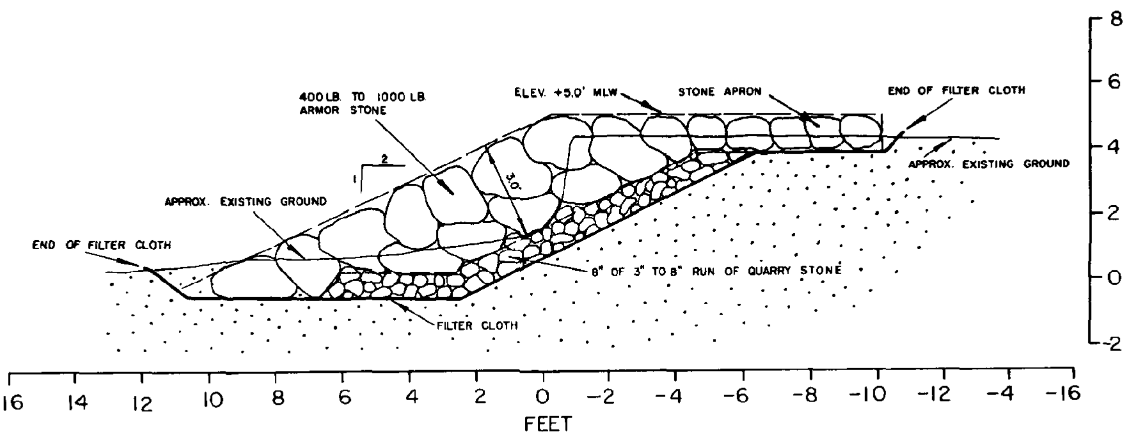
CASE 4 A STONE REVETMENT IN FRONT OF A TIMBER BULKHEAD ON UPPER HOOPER ISLAND

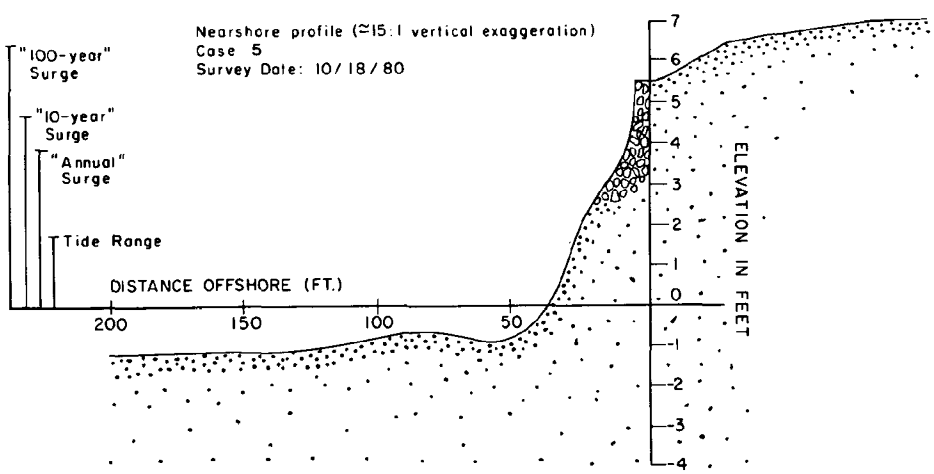
CASE 5 A STONE REVETMENT WITH TWO STONE GROINS IN TAR BAY

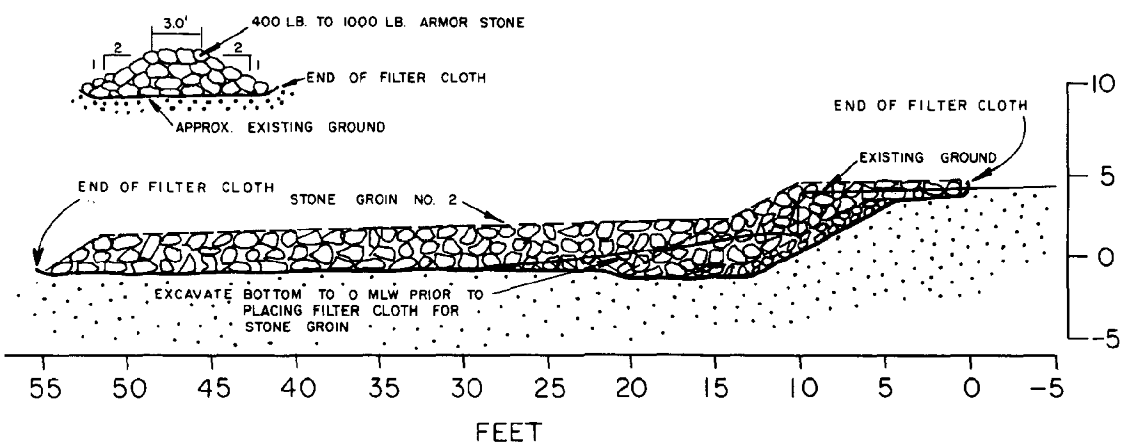
Structure was completed in 1977 at an approximate cost of $67.47/ft. for the revetment, and $50.00/ft. for the stone groins. The historical rate of erosion at the site was less than 2 ft./yr. from 1848-1942. Stone revetment, on a 2:1 slope, consists of 400-1000 lbs. stone in a 3 ft.-thick armor layer. A bedding layer, 8 in. thick, composed of 3-8 in. quarry stone, was placed beneath the armor layer. Filter material was used below the bedding layer. The revetment has a 10 ft.-wide splash apron.

Two stone groins are 62 ft. and 42 ft. long. A sand beach has accumulated and covered the toe of the revetment in the summer of 1980.

These structures are in generally good condition.



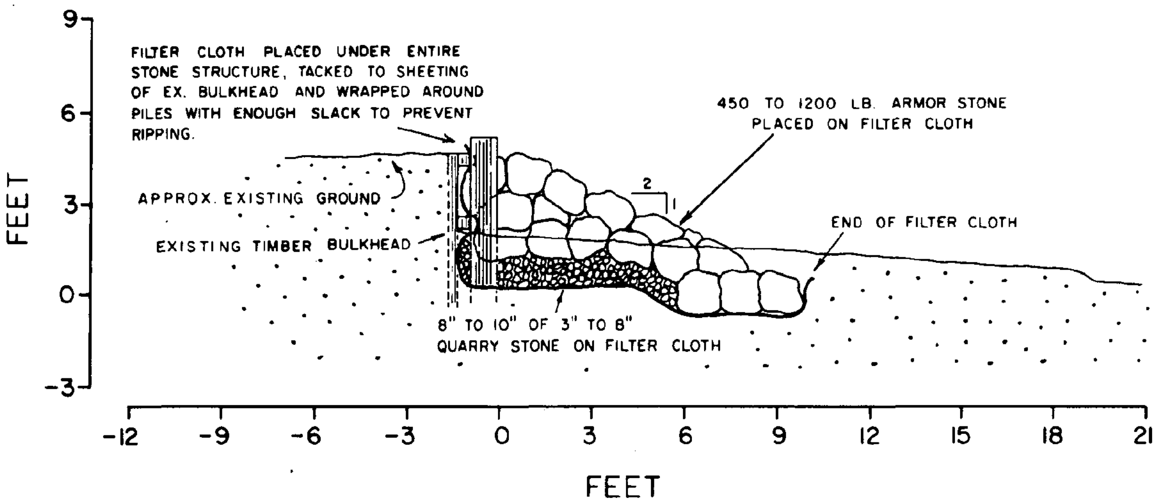


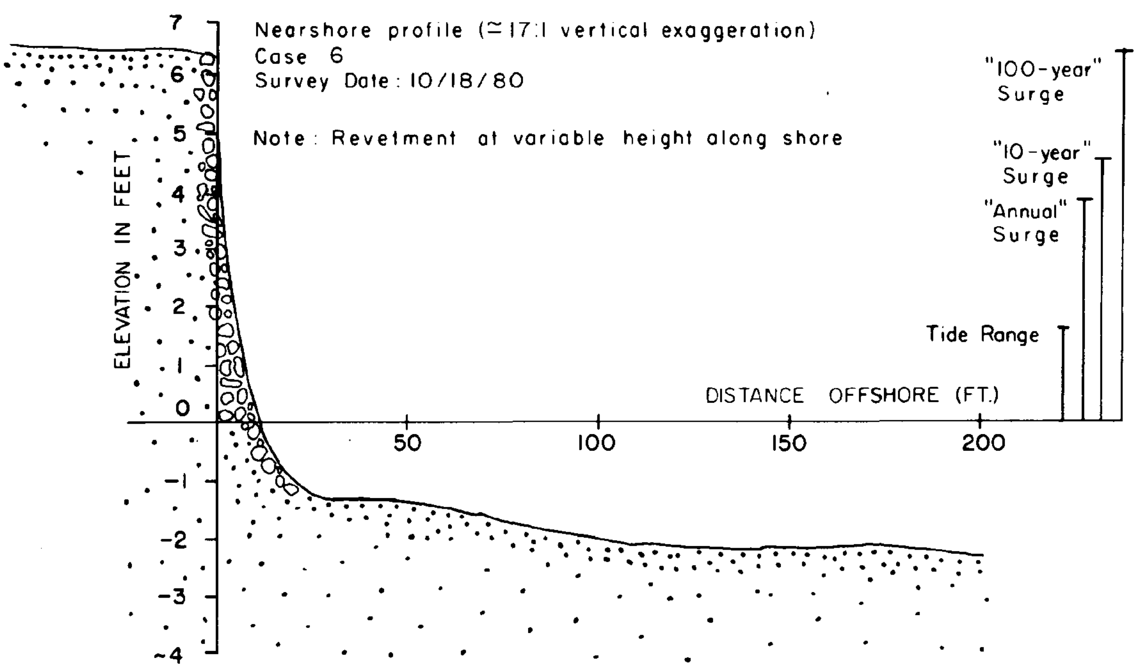


CASE 5 A STONE REVETMENT WITH TWO STONE GROINS IN TAR BAY

CASE 6 A STONE REVETMENT ON UPPER HOOPER ISLAND

Structure was completed in 1977 at a cost of $66.15/ft. The historical rate of erosion at the site was 1-2 ft./yr. from 1848-1942. Stone revetment, on approximately 2:1 slope, is in good condition. This structure fronts several properties, and is doubly-reveted in some portions. Revetment also extends onto the fastland to different heights up to 7.5 feet above MLW, in front of different properties.

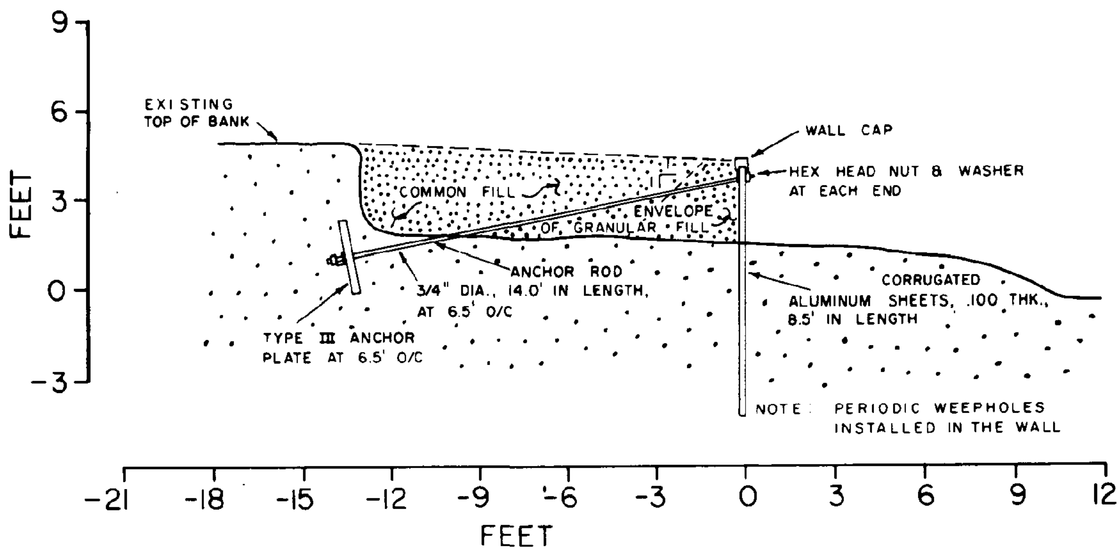


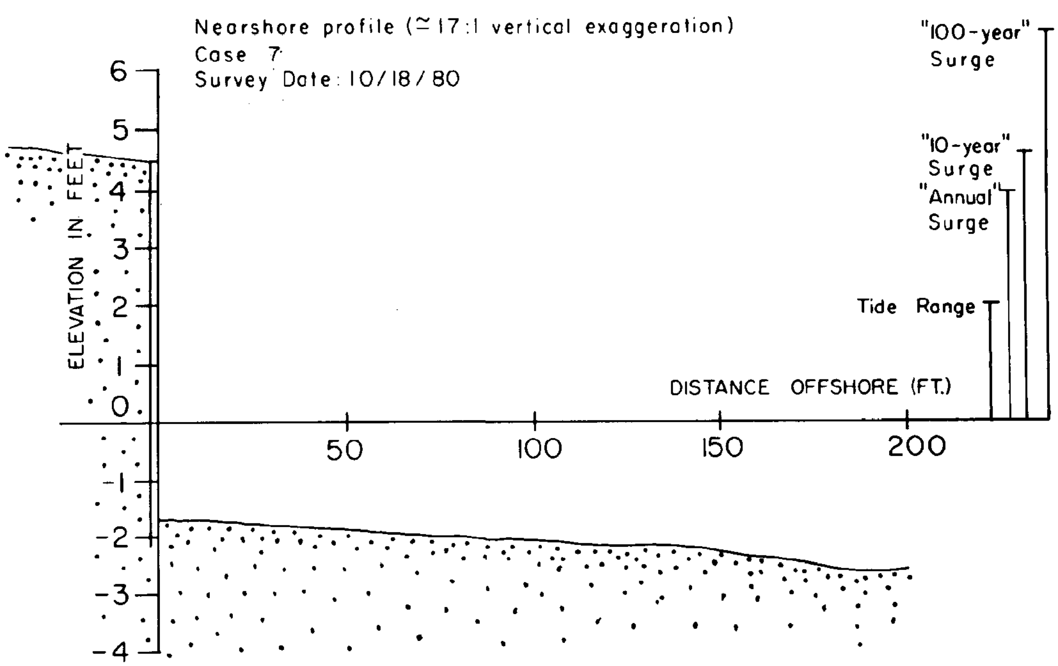


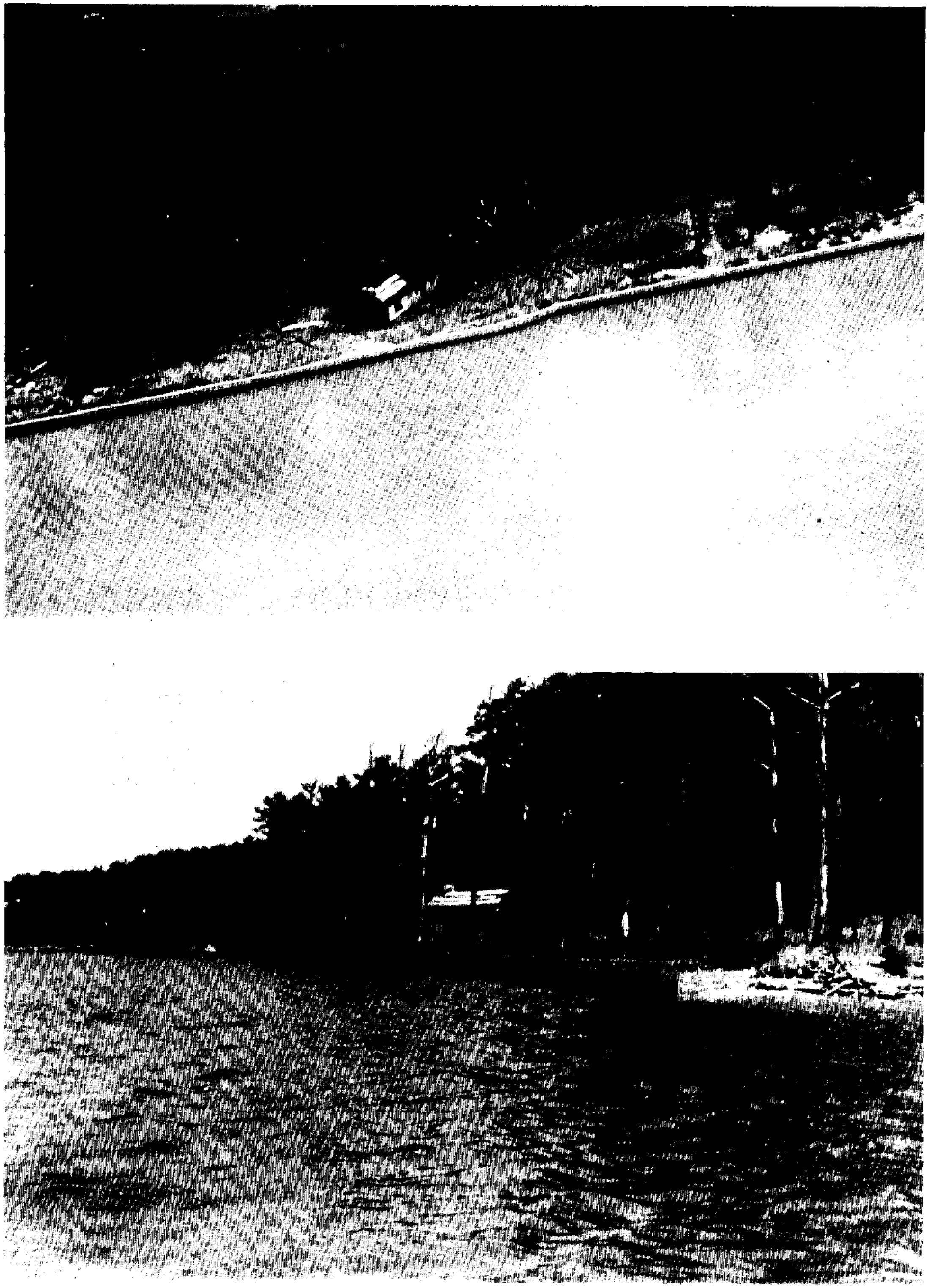
CASE 6 A STONE REVETMENT ON UPPER HOOPER ISLAND

CASE 7 AN ALUMINUM BULKHEAD AT PARKS NECK ON THE HONGA RIVER

Structure was completed in 1975 at a cost of $42.50/ft. The historical rate of erosion at the site was 5 ft./yr. from 1848-1942. Bulkhead consists of 103 aluminum corrugated panels, each 4.8 ft. wide, 0.10 in. thick, 8.5 ft. long. Short return walls of at least 14.6 ft. each used at each end have been lost due to flanking. About 50 feet of recession has occurred since emplacement of the structure. Overtopping occurs during storms, and material is being lost through the wall. An inspection report prepared in 1978 indicated (1) failure is due partly to one year delay in backfilling the structure; (2) structure exhibits lack of maintenance; (3) structure may have been partly damaged initially by heavy equipment used during backfill operation.



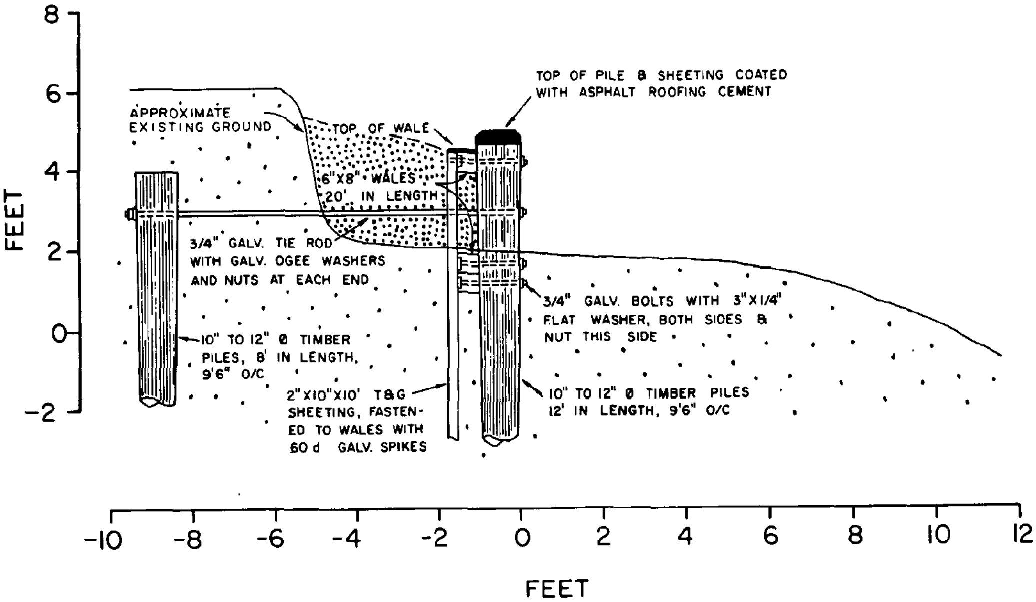


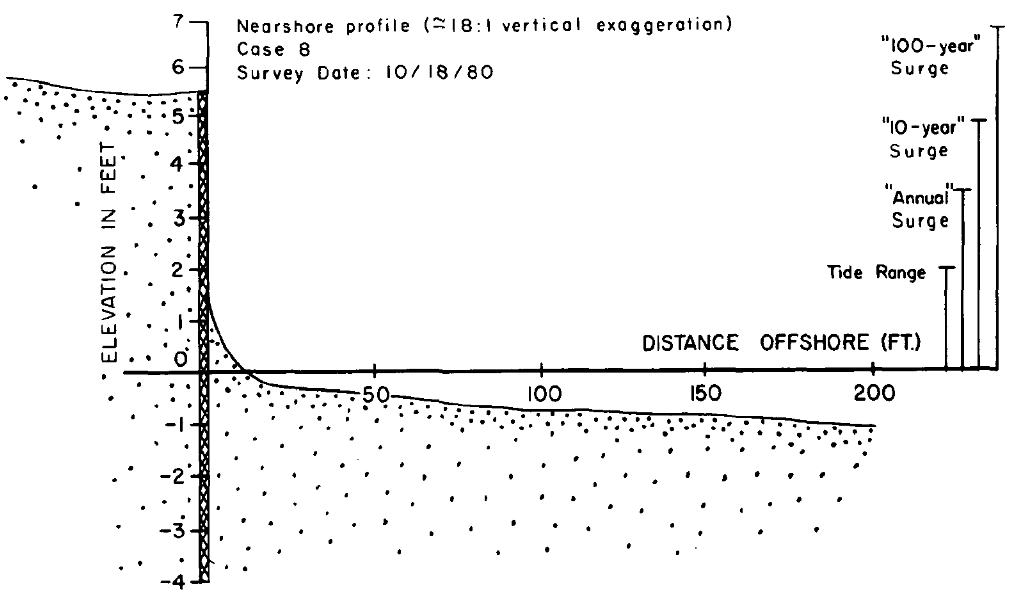


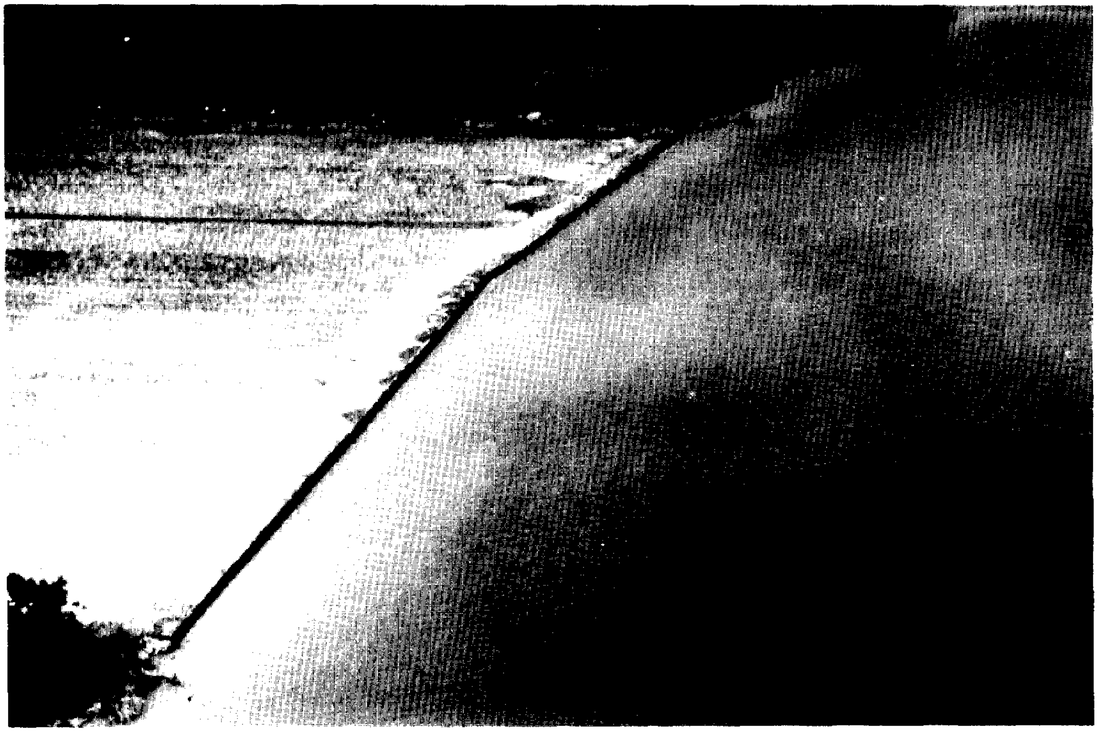
CASE 7 AN ALUMINUM BULKHEAD AT PARKS NECK ON THE HONGA RIVER

CASE 8 A TIMBER BULKHEAD ON ASQUITH ISLAND IN THE HONGA RIVER

Structure was completed in 1976 at a labor cost of $38.92/ft. All materials were provided by the property owner at an unknown cost. The historical rate of erosion at the site was 3.5 ft./yr. from 1848-1932. Bulkhead consists of 10-12 inch diameter pile, 12 ft. in length, on 9.5 ft. centers; 2X10 in. tongue-in-groove sheet pile, 10 ft. long; and deadmen with 3/4 in. galvanized tie rods on the landward side. Asphalt coating on top of piling is in poor condition. No filter material was used. At least one patch has been applied to prevent loss of material through the wall. The structure is bordered by an eroding headland on one side and an eroding shoreline on the other.



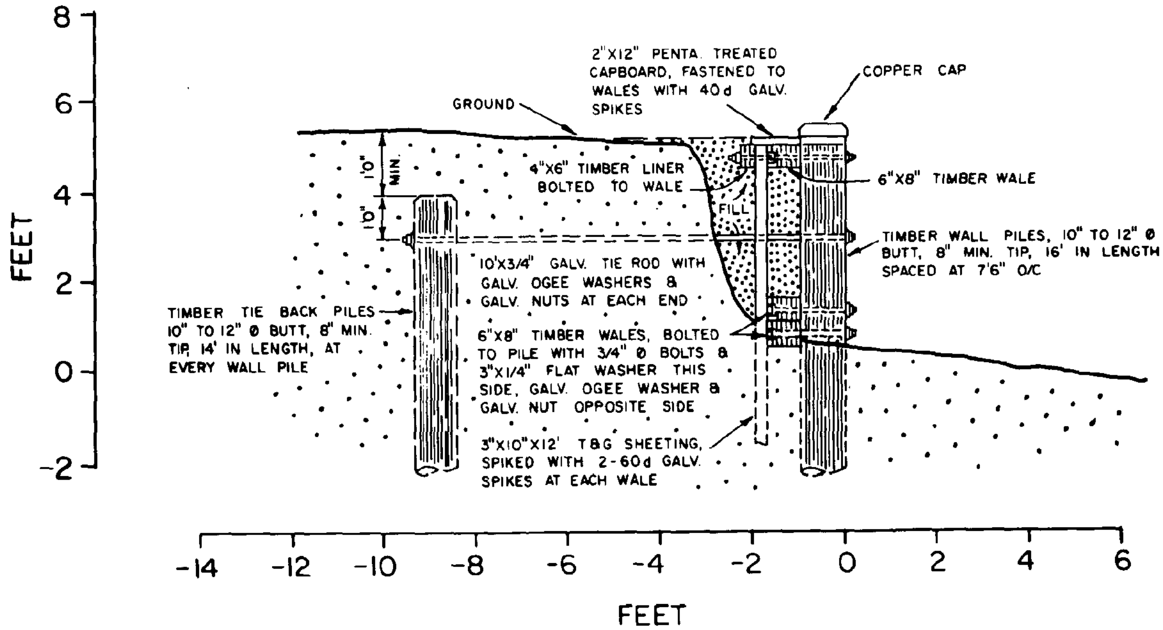


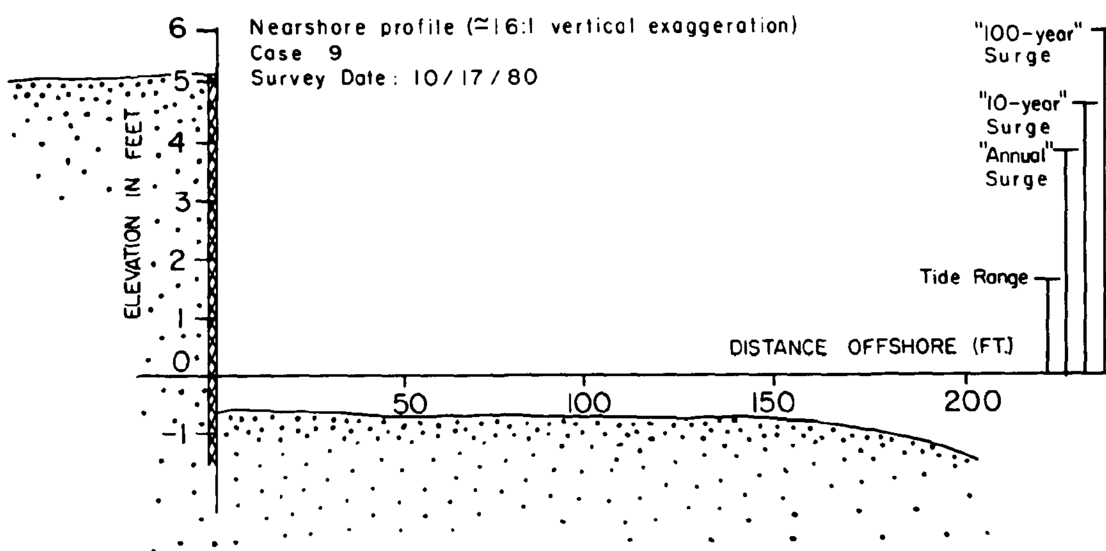


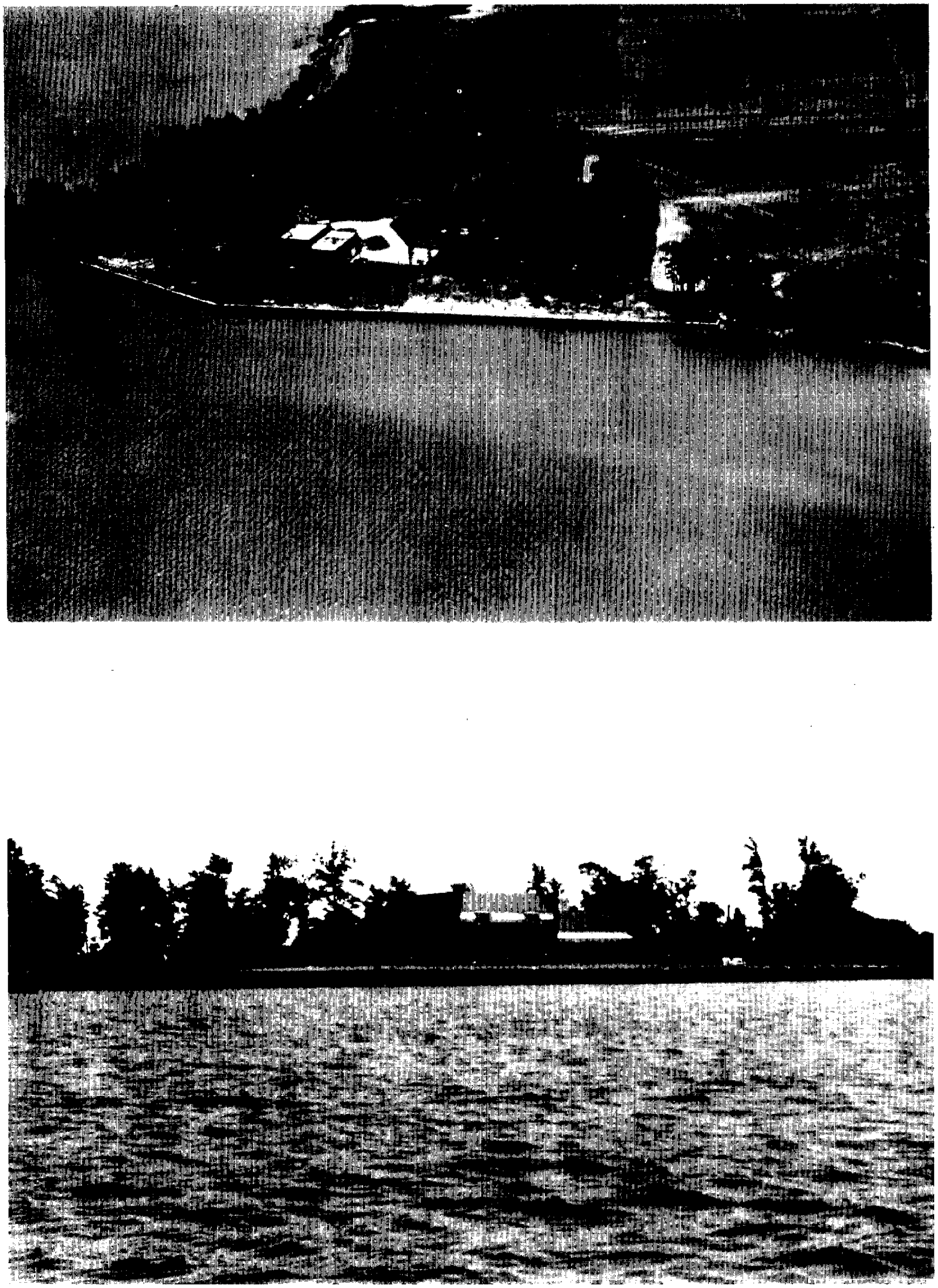
CASE 8 A TIMBER BULKHEAD ON ASQUITH ISLAND IN THE HONGA RIVER

CASE 9 A TIMBER BULKHEAD IN TRIPPE BAY

Structure was completed in 1975 at a cost of $65.43/ft. The historical rate of erosion at the site was 1-2 ft./yr. from 1847-1942. Structure consists of 16 ft. pile; 3 in. x 10 in. tongue-in-groove sheeting, 12 ft. long; and 14 ft. long deadmen, 10 ft. back, tied to the wall with 3/4 in. galvanized rod. Copper caps are installed on tops of piles. Some evidence of overtopping by waves was noticed at this site. Dead grass near bulkhead is evidence of salt damage to vegetation. Significant flanking erosion was observed at ends of the structure in the summer of 1980.



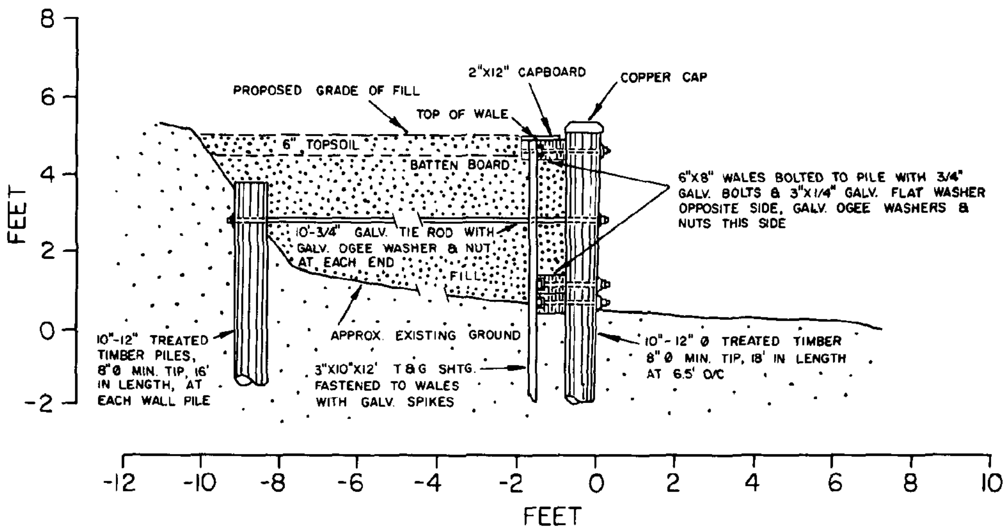


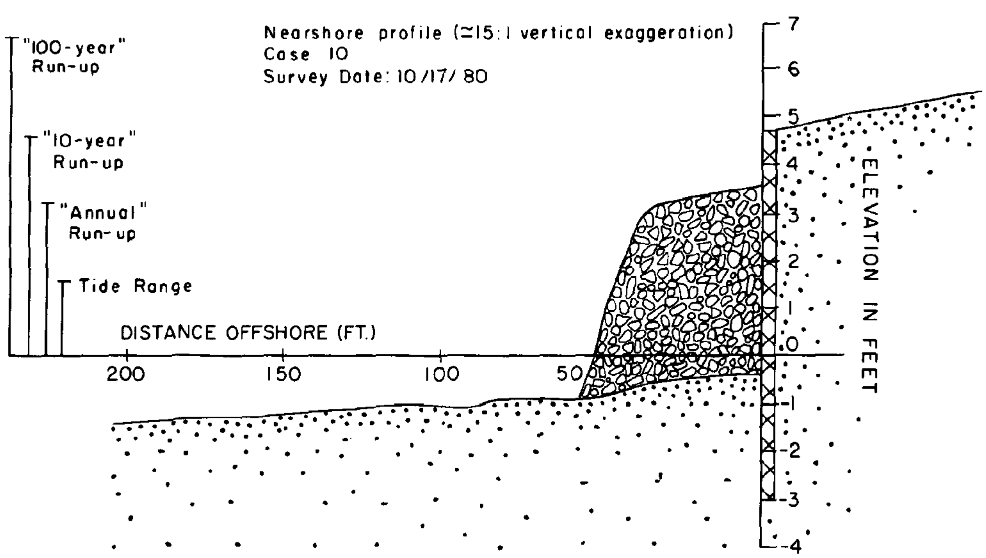


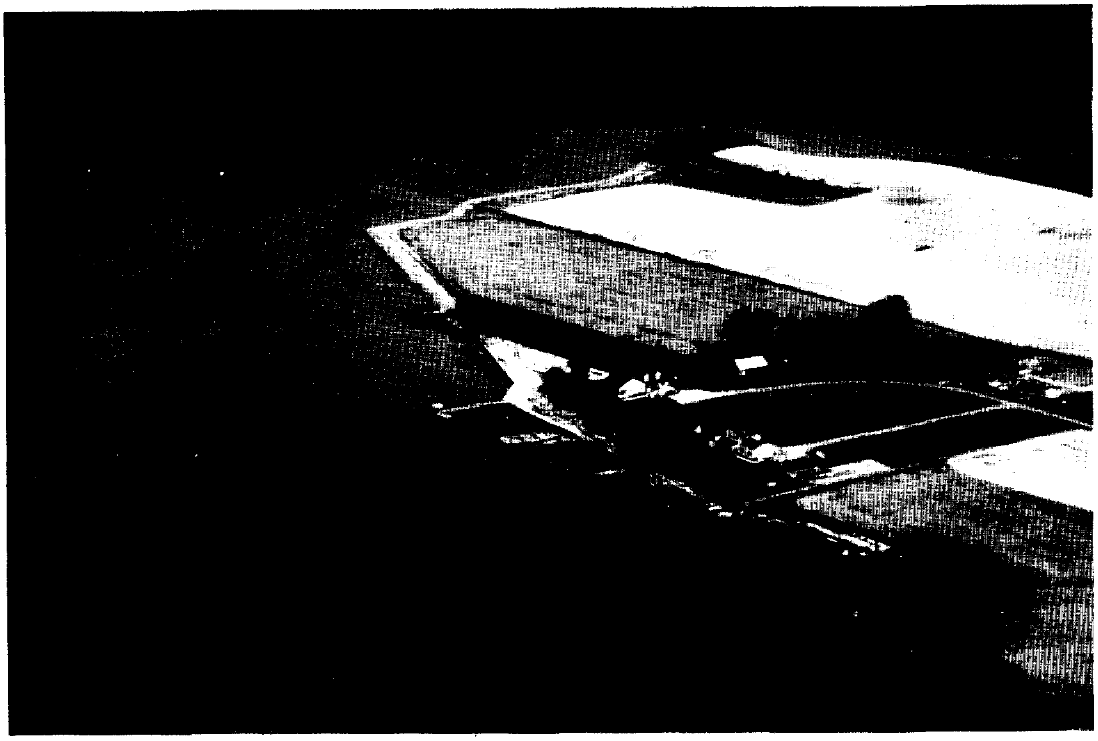
CASE 9 A TIMBER BULKHEAD IN TRIPPE BAY

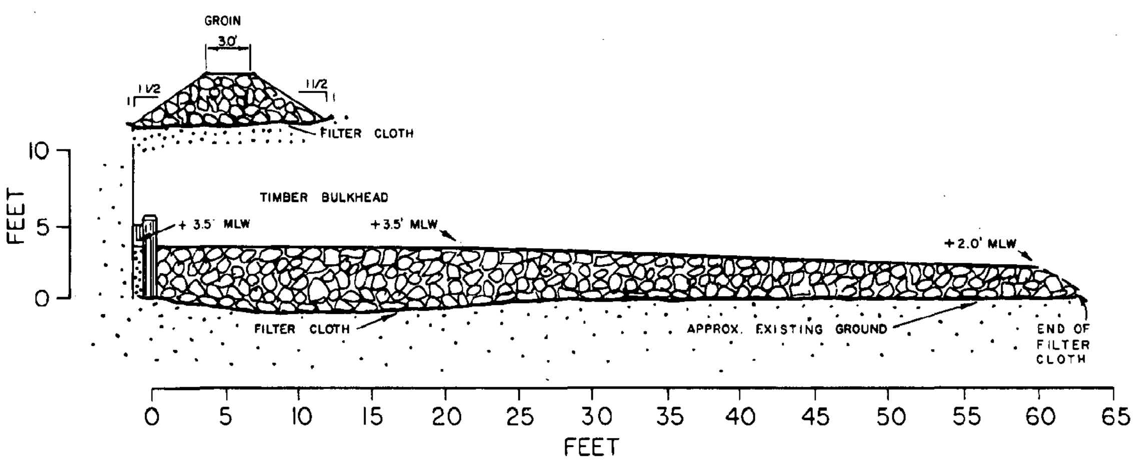
CASE 10 A TIMBER BULKHEAD ON THE SOUTH SHORE OF THE CHOPTANK RIVER WITH 2 STONE GROINS

Structure was completed in 1978 at a cost of $92.15/ft. for the bulkhead and $22.72/ft. for the groins. The historical rate of erosion at the site was 1 ft./yr. from 1847-1942. Bulkhead consists of 10-12 in. diameter pile, 18 ft. long; 3 in. x 10 in. x 12 ft. tongue-in-groove creosoted sheetpile; and 2 wales. Deadmen are 16 ft. long. Piles are covered with 16 oz. sheet copper cap. Two stone groins are approximately 50 ft. long. Fillet had formed on the left sides in the summer of 1980. Natural grass is growing on the beach to the left of the groins. There is evidence of splashover at this site.





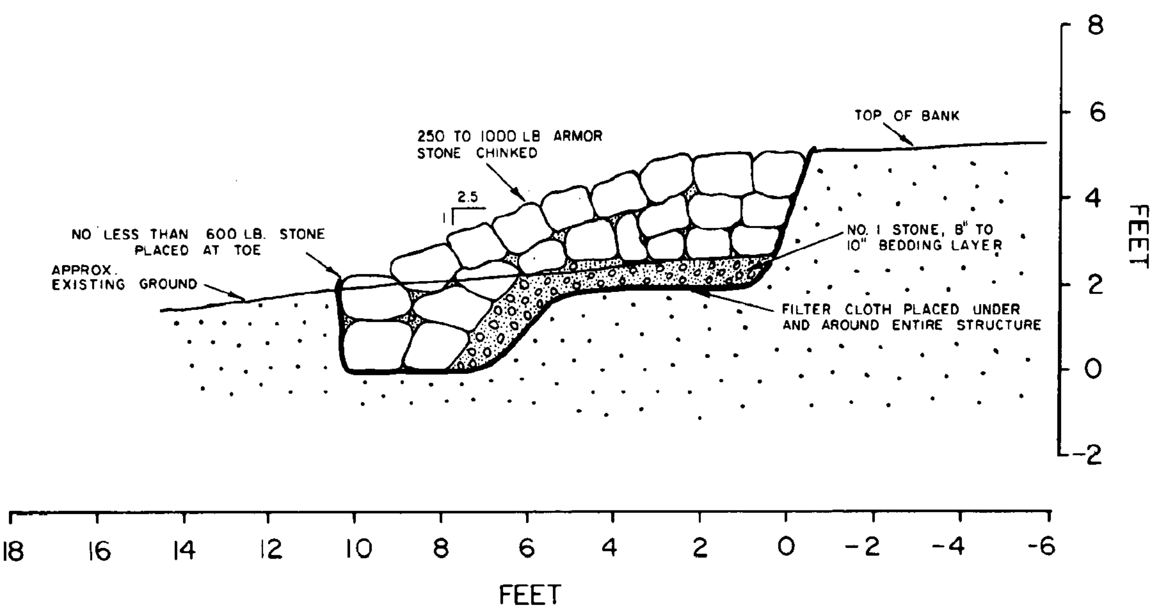


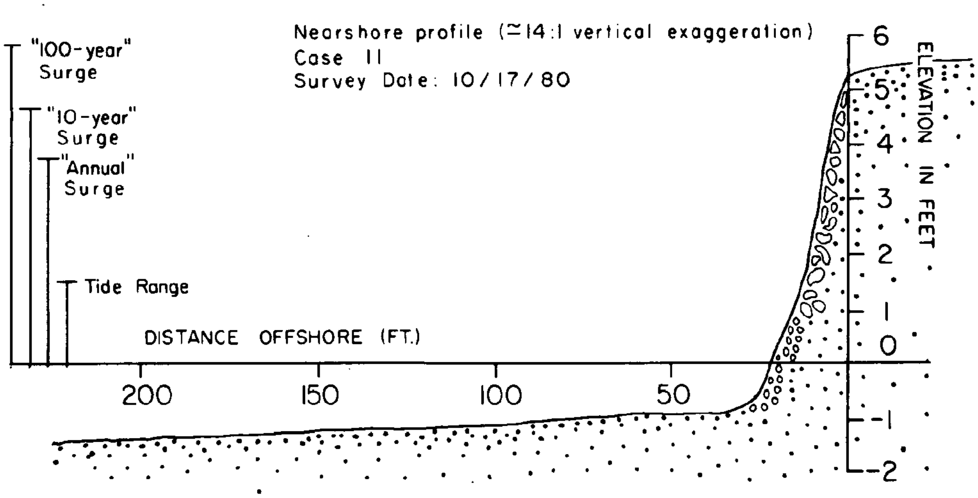


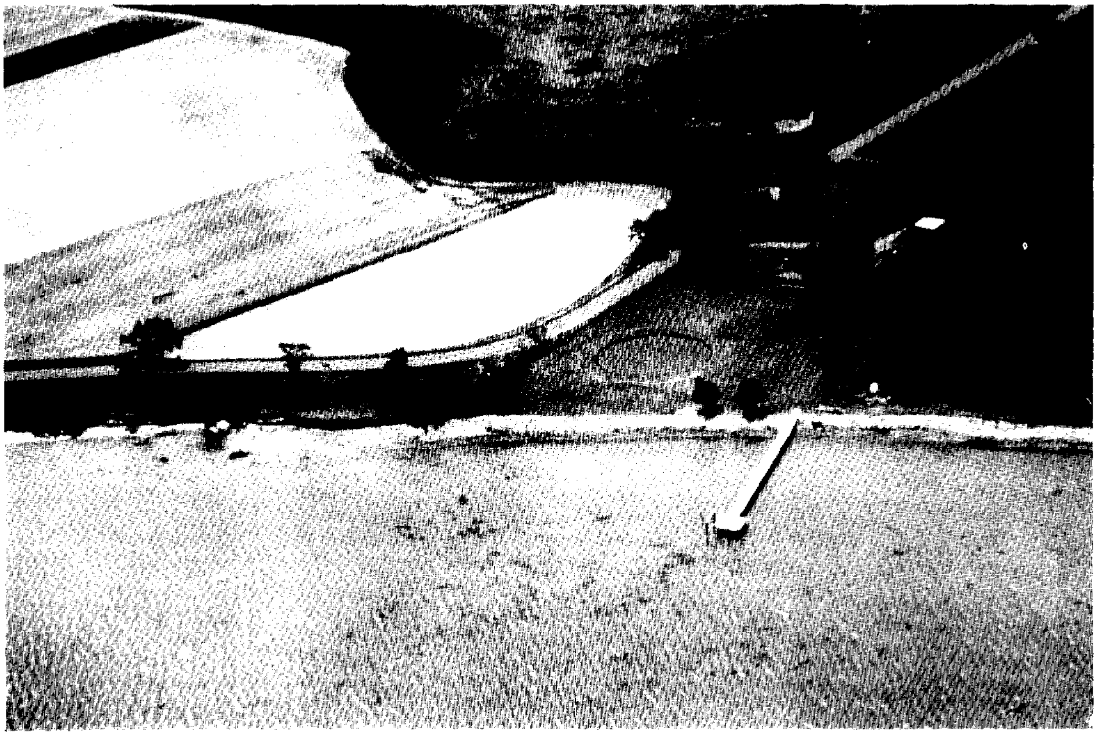
CASE 10 A TIMBER BULKHEAD ON THE SOUTH SHORE OF THE CHOPTANK RIVER WITH 2 STONE GROINS

CASE 11   A STONE REVETMENT IN BRANNOCK BAY

Structure was completed in 1975 at a cost of $25.00/ft. The historical rate of erosion at the site was 4 ft./yr. from 1847-1942. Stone revetment, on a 2.5:1 slope, consists of 250-1000 lbs. stone in a 2-3.5 ft.-thick armor layer. A 3 ft.-wide splash apron was also installed. This structure is in generally good condition; but, there is active flanking erosion alongshore, as evidenced by a 4.3 ft. scarp in the adjoining exposed clay bank.





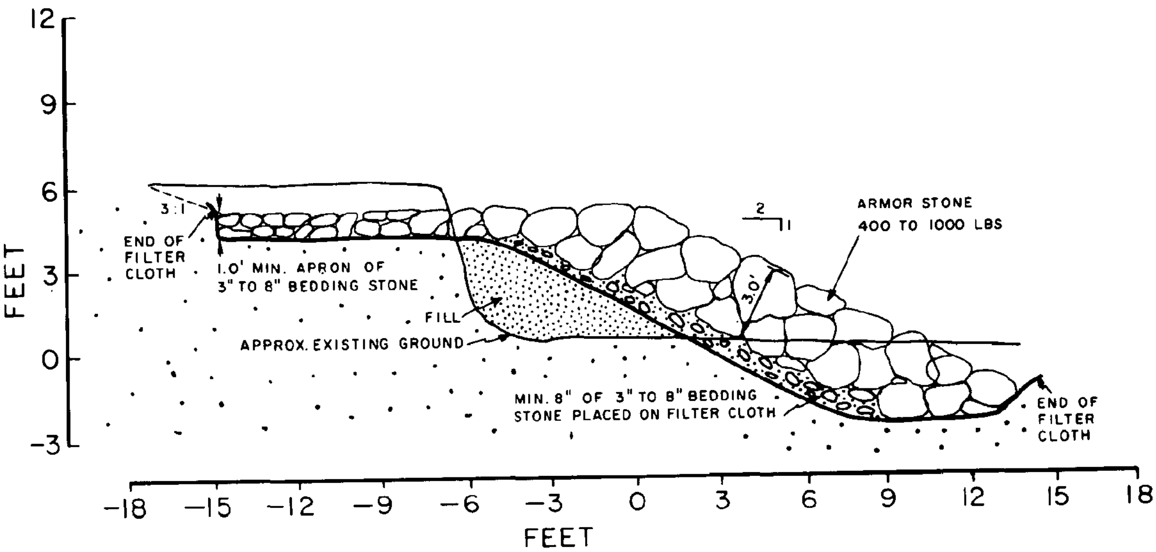


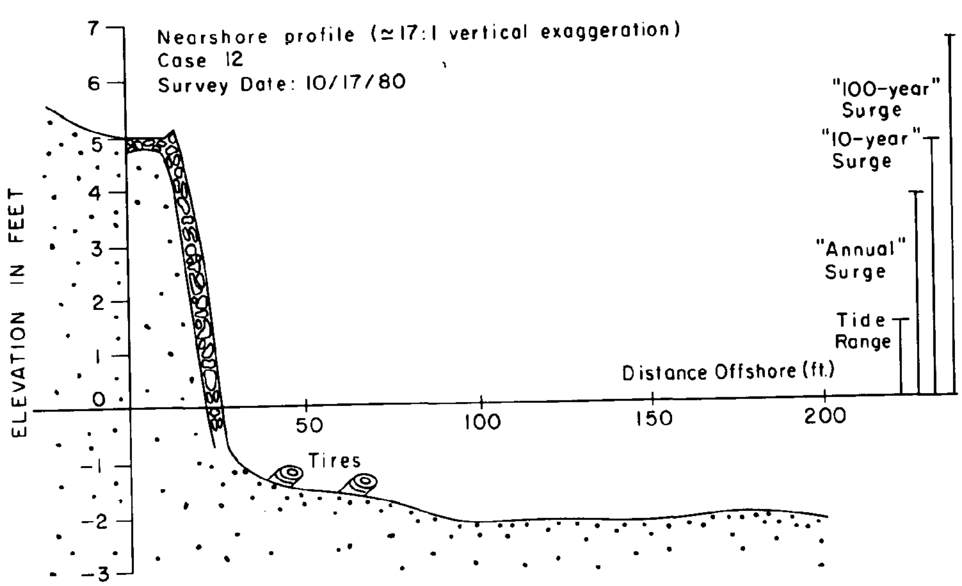
CASE 11 A STONE REVETMENT IN BRANNOCK BAY

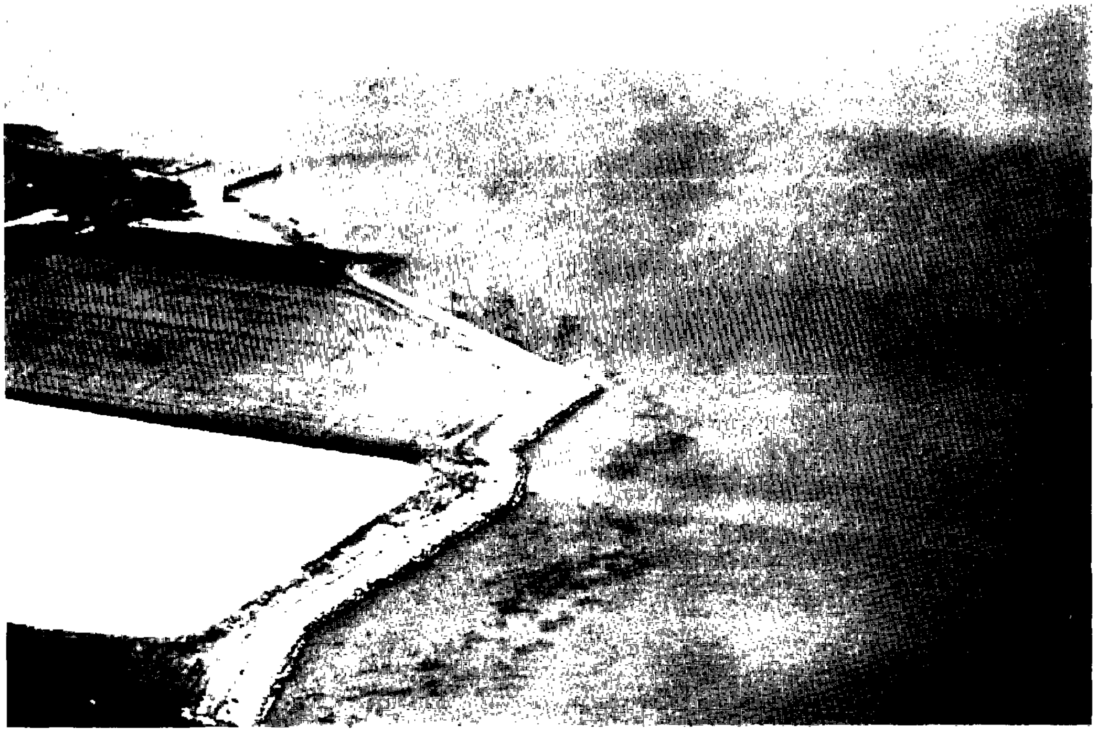
CASE 12 A STONE REVETMENT ON THE SOUTH SHORE OF THE CHOPTANK RIVER

Structure was completed in 1977 at a cost of $61.00/ft. The historical rate of erosion at the site was less than 1-2 ft./yr. from 1847-1932. Stone revetment, on a 2:1 slope, consists of 400-1000 lbs. stone in a 3 ft.-thick armor layer. Bedding layer is 8 in. thick, composed of 3-8 in. stone. Filter cloth was used below the bedding layer.

This structure is in generally good condition. The ties located offshore appear to have been used in a breakwater consisting of regularly-spaced pilings which held the tires. Automobiles and an old cement truck were also placed in the nearshore to control erosion.







CASE 12 A STONE REVETMENT ON THE SOUTH SHORE OF THE CHOPTANK RIVER

C. Cases along the Lower Western Shore

This area includes portions of St. Mary's County along the Chesapeake Bay and lower Potomac River (Figures 2.4, 2.5). The sections below present a brief physical description of the shoreline and coastal processes, followed by discussion of the case studies.

St. Mary's County The Chesapeake Bay shoreline in St. Mary's County runs from the Patuxent River mouth to Point Lookout. The northern portion of this reach contains the Patuxent Naval Air Station, and the shoreline at the southern end is within the boundary of Point Lookout State Park. Shorefront areas in between contain heavily-wooded lands, fields, and scattered residential development protected at different points by erosion-control structures.

There are exposed eroding banks in many areas which range in height from 3-8 feet. The beaches on the shoreline profiles are of varying widths; in a few cases, there are well-developed berms on the summer beach profiles with grasses or small shrubs stabilizing the sand.

A portion of the shoreline included in the study along the lower Potomac River contains wide beaches composed of coarse sand and gravel. Near Point Lookout and Piney Point, shorefront homes are separated from the water by a wide vegetated buffer strip of beach and berms. Elsewhere woodlands and fields are perched on exposed eroding banks next to the beach.

Coastal Processes Many of the historical erosion rates for the lower Western Shore are less than 4 feet/year (see Chapter V). The mean tide range in the area is about 1.0 feet. The storm surges from "annual" storms are about 3 feet, and the surges from "100-year" storms can be greater than 4.5 feet above mean low water. Waves during these severe storms can be as high as 3-5 feet on top of the storm surges, depending on the type of storm and its orientation relative to the fetches in the area.

Waves in the area approach from the west, south, and northeast with the longest fetches. The annual wave energy on the shoreline is higher in this area than in many other areas farther north on the Chesapeake Bay. Shorefront areas exposed to the long fetch from the south are particularly susceptible to wave attack from tropical storms.

The wave and storm conditions are discussed in greater detail along with the other coastal processes in Chapter V.

Case Studies The case studies for structures selected in this area include:

Case No.	Structure
• 13	Stone revetment near Point Lookout (1564 feet in length).
• 14	A timber bulkhead at Tall Timbers on the Potomac River (1204 feet in length).

The following pages present brief descriptions of each structure, and nearshore bottom profiles collected at the sites.

Next Pages: Figure 2.4. Shoreline along the lower western shore of the Chesapeake Bay in Maryland.

Figure 2.5. Some representative shoreline profiles collected in the summer of 1980 along the lower western shore of the Chesapeake Bay in Maryland.

Figure 2.4

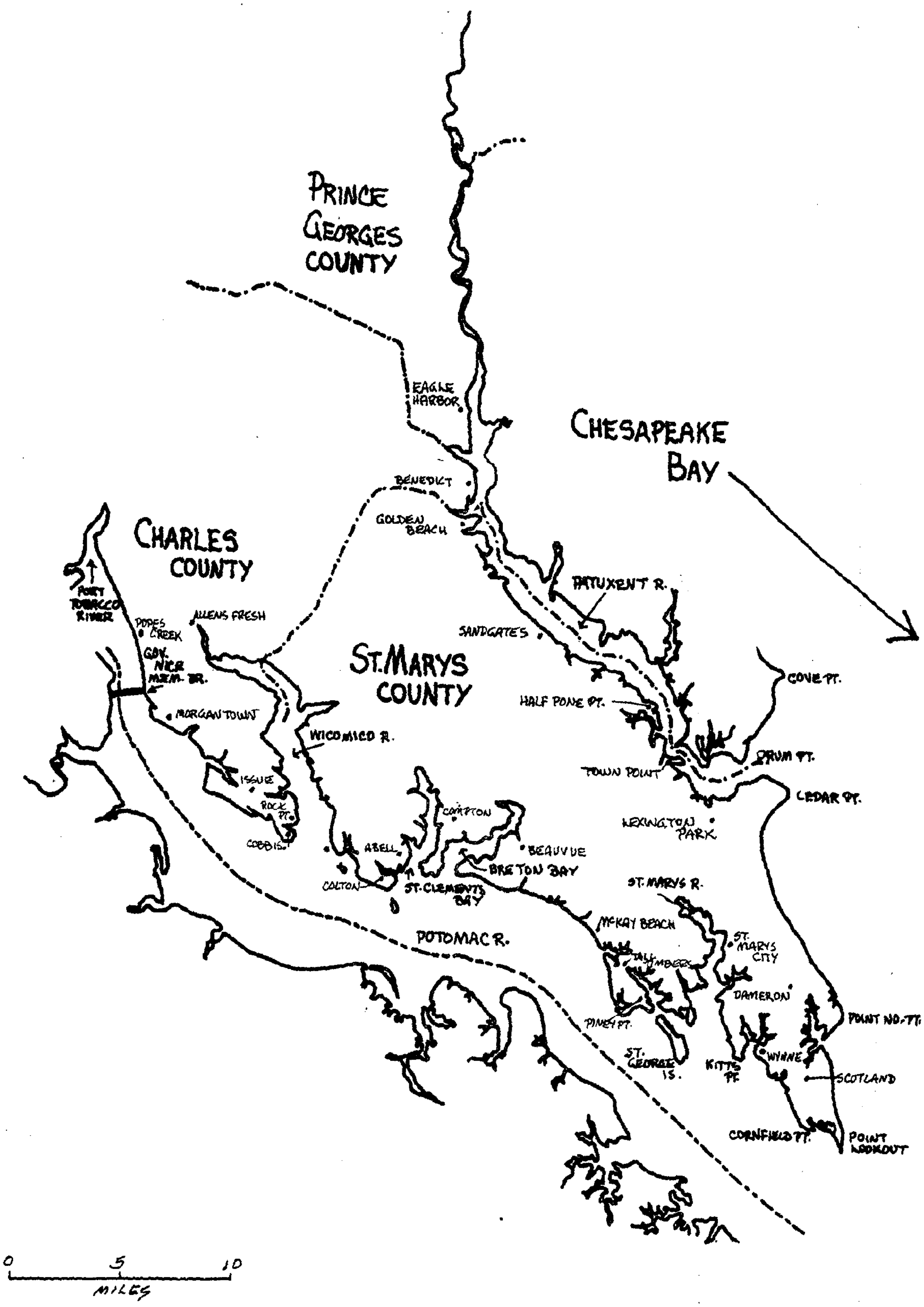
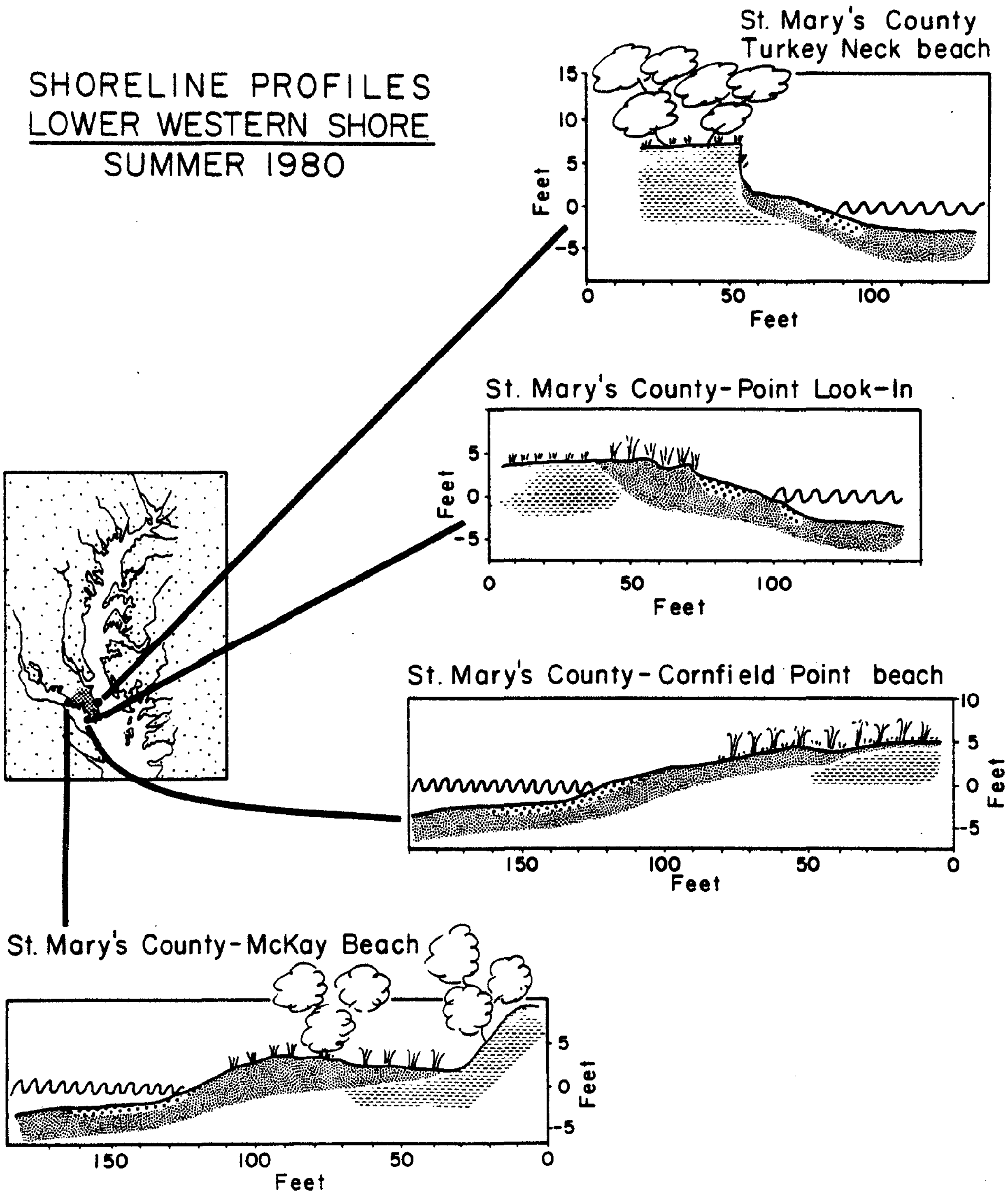

PRINCE GEORGES COUNTY
EAGLE HARBOR
CHESAPEAKE BAY
BENEDICT
GOLDEN BEACH
CHARLES COUNTY
PORT TOBACCO RIVER
POPES CREEK
ALLENS FRESH
GOV. NICE MEM. BR.
MORGAN TOWN
PATUXENT R.
SANDGATES
ST. MARYS COUNTY
HALF PONE PT.
COVE PT.
WICOMICO R.
TOWN POINT
DRUM PT.
ISSUE
ROCK PT.
COBB IS.
CEDAR PT.
COMPTON
LEXINGTON PARK
ABELL
BEAUVUE
BRETON BAY
COLTON
ST. CLEMENTS BAY
ST. MARYS R.
McKAY BEACH
POTOMAC R.
ST. MARYS CITY
DAMERON
PINEY PT.
POINT NO PT.
ST. GEORGE IS.
KITTS PT.
WYNNE
SCOTLAND
CORNFIELD PT.
POINT LOOKOUT
0 5 10
MILES

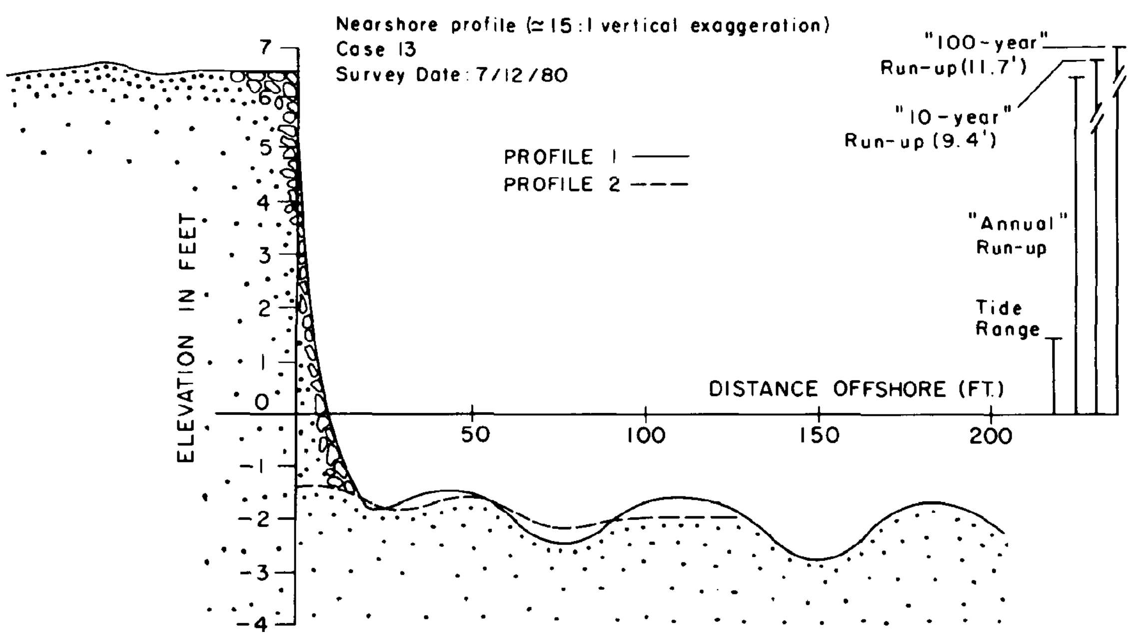
Figure 2.5

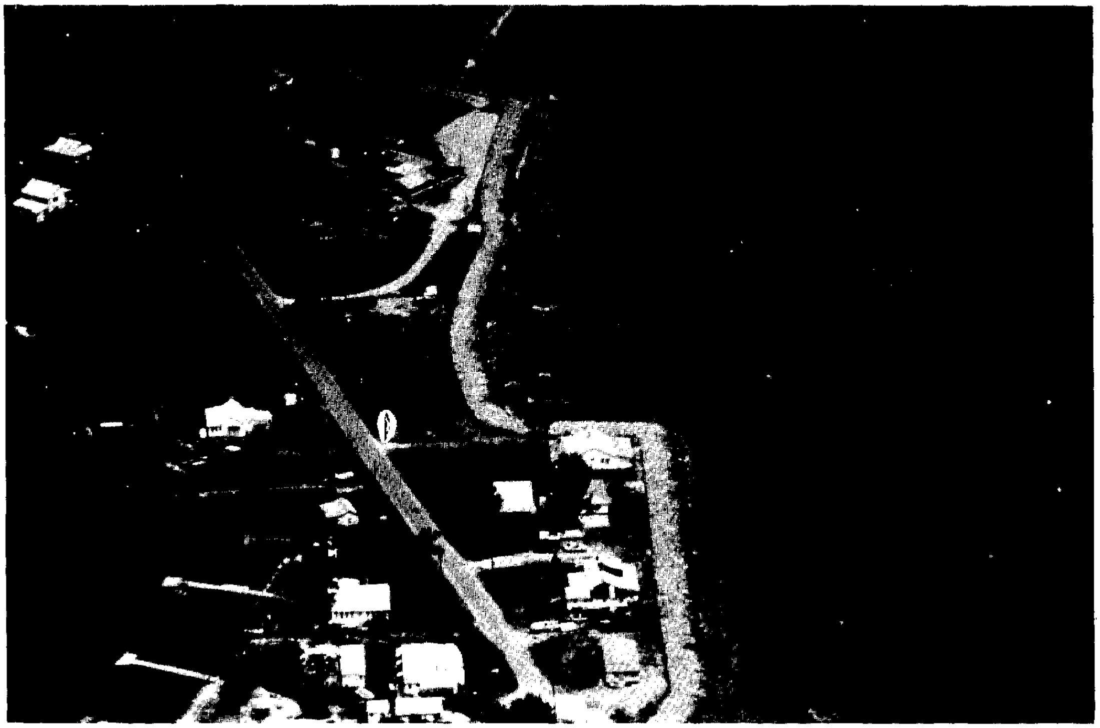

SHORELINE PROFILES
LOWER WESTERN SHORE
SUMMER 1980
St. Mary's County
Turkey Neck beach
15
10
5
0
-5
Feet
0
50
100
Feet
St. Mary's County-Point Look-In
5
0
-5
Feet
0
50
100
Feet
St. Mary's County-Cornfield Point beach
10
5
0
-5
Feet
150
100
50
0
Feet
St. Mary's County-McKay Beach
5
0
-5
Feet
150
100
50
0
Feet

CASE 13 A STONE REVETMENT NEAR POINT LOOKOUT

Structure was completed in 1974 at a cost of $132.96/ft. The historical rate of erosion at the site was 5-14 ft./yr. from 1849-1942. Stone revetment on a 2:1 slope consists of 1400-2800 lbs. stone in a 4 ft.-thick armor layer. A bedding layer, 1 ft. thick, of smaller stone was placed below the armor layer. Filter material was used below the bedding layer. The revetment has a 20 ft.-wide splashover apron.

This structure is in generally good condition. There is a receding shoreline alongshore to the north of the structure. Eighty-five feet of shoreline recession has occurred there since the structure completion date.

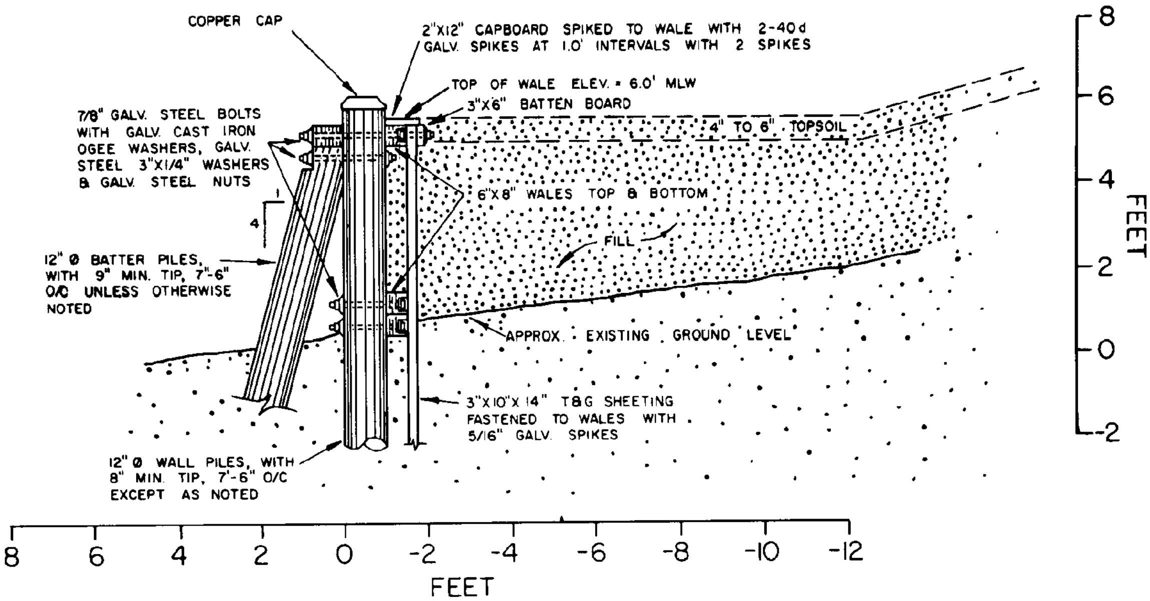


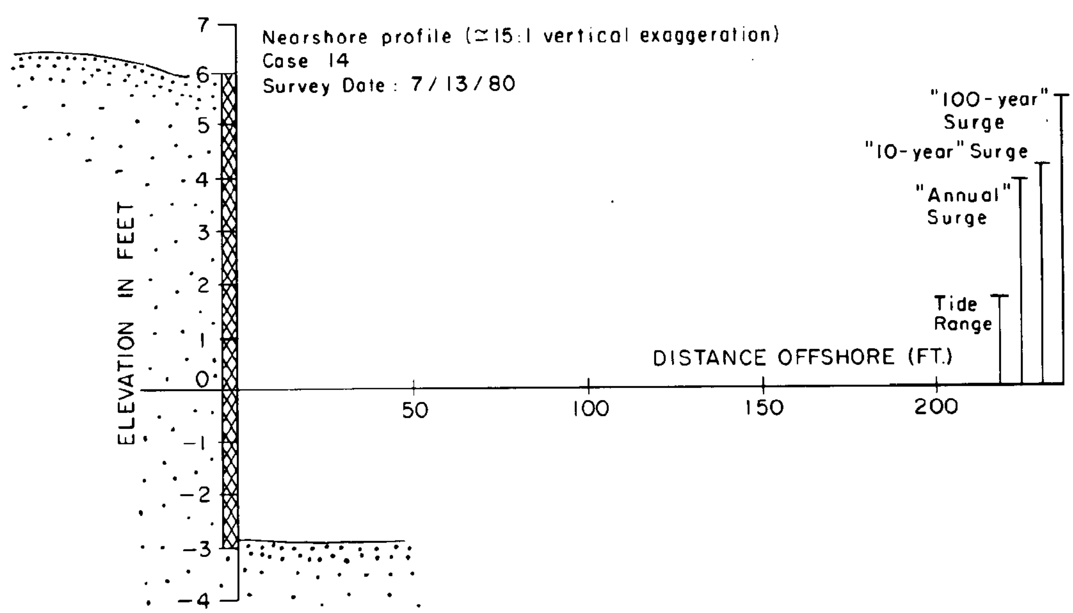


CASE 13 A STONE REVETMENT NEAR POINT LOOKOUT

CASE 14   A TIMBER BULKHEAD AT TALL TIMBERS ON THE POTOMAC RIVER

Structure was completed in 1976 at a cost of $112.19/ft. The historical rate of erosion at the site was 1.5 ft./yr. from 1868-1968. Bulkhead consists of a wooden sheetpile with an offshore batter pile. Two stone groins (not part of this project) are trapping small amounts of sand, but no beach exists in front of the bulkhead. One of these groins, 80 ft.-long, was added in 1979 at a cost of $156.00/ft. to act as a breakwater intended to reduce splashover. The bulkhead is in generally good condition. There is evidence of splashover at the site, and the bulkhead has flanking erosion alongshore at one end.





CASE 14 A TIMBER BULKHEAD AT TALL TIMBERS
ON THE POTOMAC RIVER

D. Cases along the Calvert County and Lower Anne Arundel County shorelines

This area contains the shoreline between the Patuxent River mouth and the Chesapeake Bay Bridge (Figures 2.6 and 2.7). The sections below present a brief physical description of the shoreline and coastal processes, followed by a discussion of the case studies which were selected from this area.

SHORELINE DESCRIPTIONS

Calvert County The Calvert County shoreline along the Chesapeake Bay is composed mainly of large bluffs, higher than 50 feet in many areas, which extend for several thousand feet at a stretch along the water's edge. The bluff faces are mostly exposed and eroding, but they are covered with vines and shrubs in a few places. Sections of the bluffs are separated by ravines and stream valleys which contain either woodlands or marsh.

The beaches at the base of these bluffs are of varying widths and may contain small berms on the summer shoreline profiles. At Cove Point and near Long Beach, the beach is separated from the bluffs by a wide flat terrace which contains trees and open grassy areas.

Most of the shorefront bluffs adjacent to the main Chesapeake Bay in Calvert County are heavily-wooded, with scattered residential development in among the trees. More concentrated residential development protected by shoreline structures can be found at the communities shown on the map. Houses in these areas are located both along the bluffs, and on the low berm that extends landward immediately adjacent to the beach.

Above the Town of Chesapeake Beach in Calvert County, the shoreline is composed of low banks ranging in height from 3-12 feet. Much shorefront residential development is present, and erosion-control structures form a nearly continuous network along the water's edge in some spots. The few unprotected areas contain beaches at the base of the shoreline banks, and marshes.

Lower Anne Arundel County The lower Anne Arundel County shoreline extends from Herring Bay to the Chesapeake Bay Bridge. This area is characterized by gently rolling hills between 15 and 80 feet high. In Herring Bay, and in the lower reaches of the West, Rhode, and Severn Rivers, the hillsides end at the shoreline in steep, exposed, eroding bluffs and high banks. The beach at the base of these hillsides is narrow or absent, and trees growing along the land's edge are often falling off the shoreline banks into the water.

In most other shorefront areas of lower Anne Arundel County, the steep hillsides are covered with trees and shrubs, or they slope gently down to the waterline. On isolated points of land, the beach may be separated from higher ground by a wide low terrace containing trees, grassy areas, or marsh. Pocket marshes are also present in protected coves. Elsewhere, a beach of varying width is present on the shoreline profile, and the beach sands are stabilized by shrubs and grasses in many locations.

Next Pages: Figure 2.6. Shoreline along Calvert and lower Anne Arundel County on the Chesapeake Bay in Maryland.

Figure 2.7. Some representative shoreline profiles collected in the summers of 1980 and 1981 along the Calvert and lower Anne Arundel County shoreline.

Figure 2.6

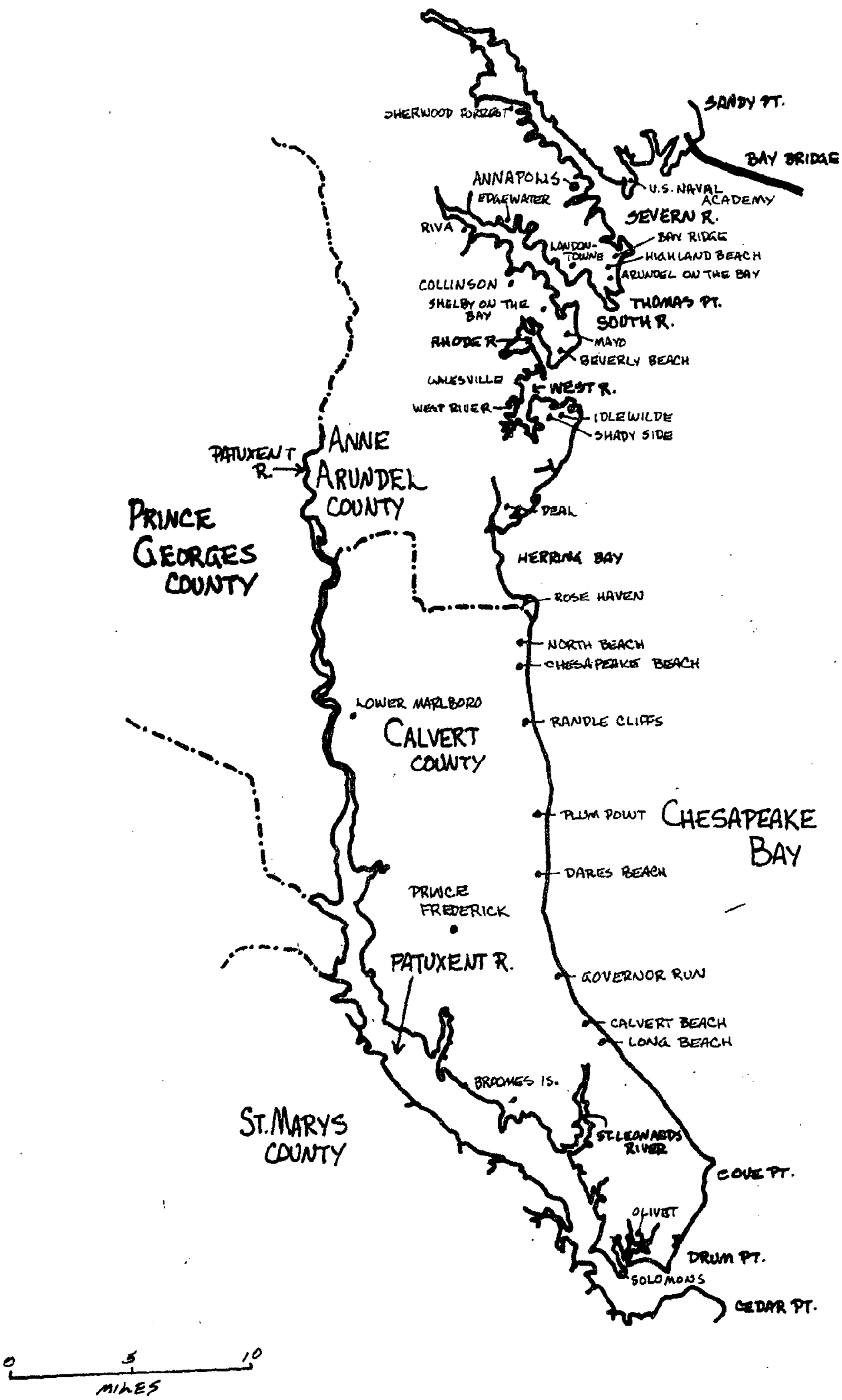
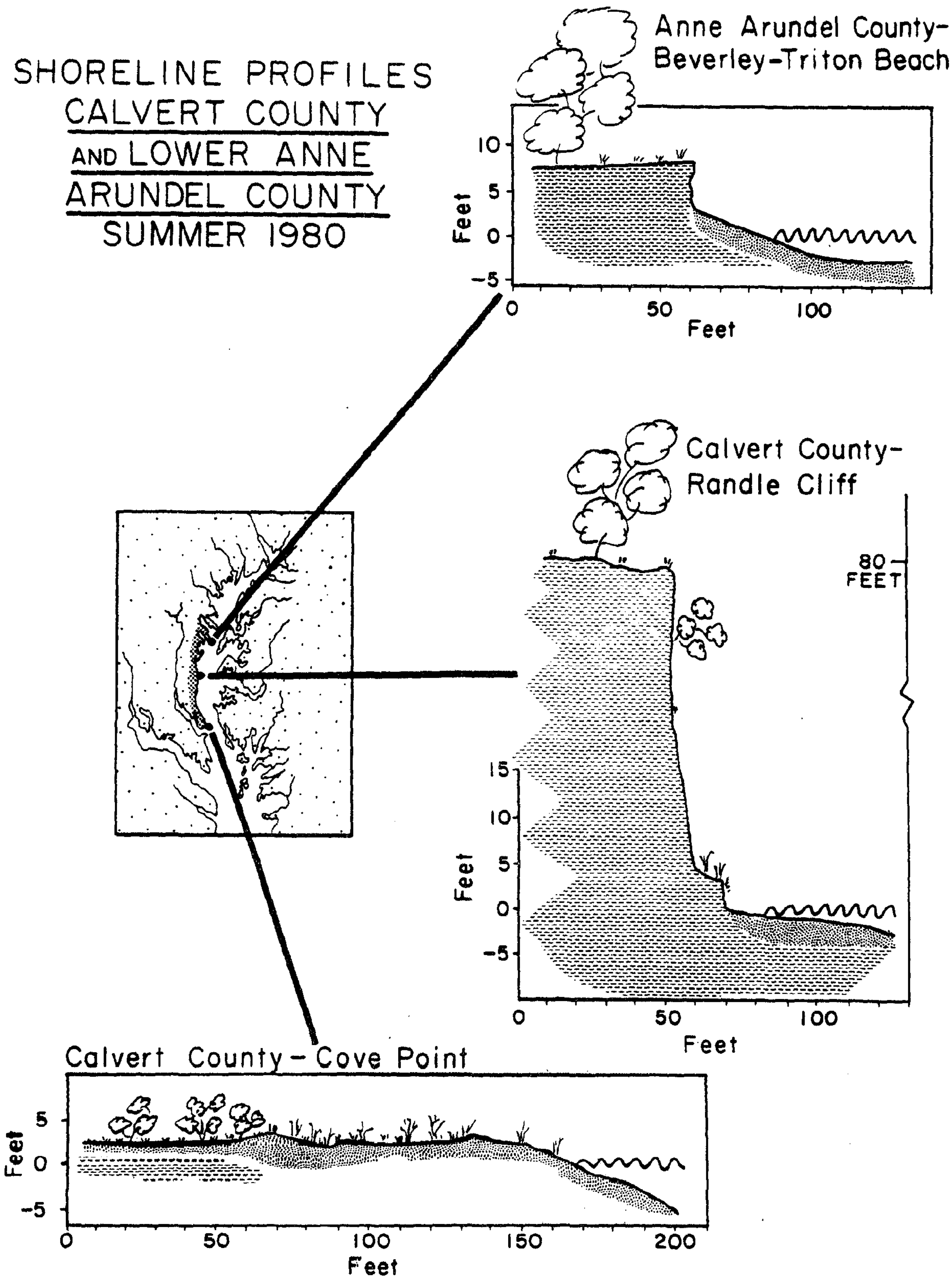

SANDY PT.
BAY BRIDGE
SHERWOOD FORREST
ANNAPOLIS
EDGEWATER
U.S. NAVAL ACADEMY
SEVERN R.
RIVA
LONDON-TOWNE
BAY RIDGE
HIGHLAND BEACH
ARUNDEL ON THE BAY
COLLINSON
SHELBY ON THE BAY
THOMAS PT.
SOUTH R.
RHODE R.
MAYO
BEVERLY BEACH
GALESVILLE
WEST R.
WEST RIVER
IDLEWILDE
SHADY SIDE
PATUXENT R.
ANNE ARUNDEL COUNTY
PRINCE GEORGES COUNTY
DEAL
HERRING BAY
ROSE HAVEN
NORTH BEACH
CHESAPEAKE BEACH
LOWER MARLBORO
RANDLE CLIFFS
CALVERT COUNTY
PLUM POINT
CHESAPEAKE BAY
DARES BEACH
PRINCE FREDERICK
PATUXENT R.
GOVERNOR RUN
CALVERT BEACH
LONG BEACH
BROOMES IS.
ST. MARYS COUNTY
ST. LEONARDS RIVER
COVE PT.
OLIVET
DRUM PT.
SOLOMONS
CEDAR PT.
0
5
10
MILES

Figure 2.7



SHORELINE PROFILES
CALVERT COUNTY
AND LOWER ANNE
ARUNDEL COUNTY
SUMMER 1980
Anne Arundel County-
Beverley-Triton Beach
10
5
0
-5
Feet
0
50
100
Feet
Calvert County-
Randle Cliff
80
FEET
15
10
5
0
-5
Feet
0
50
100
Feet
Calvert County - Cove Point
5
0
-5
Feet
0
50
100
150
200
Feet

Most communities contain concentrated residential shorefront development. Some houses have landscaped hillsides leading down to the water. Many houses are protected by erosion-control structures which often form a nearly-continuous network along the water's edge.

Between Annapolis and the Bay Bridge, the terrain in shorefront areas on the Bay is noticeably flatter, and shoreline banks are generally less than 10 feet high. The banks are covered with trees and hedgerows in some areas, and contain structures in other areas. Shorefront residential development is found principally in protected coves. Most houses in these areas are separated from the water by a wide vegetated buffer strip.

Coastal Processes Many of the shoreline reaches in Calvert and lower Anne Arundel County possess historic erosion rates between 2 and 8 feet per year. There are a few sites where the shoreline is stable or accreting. The shoreline sediments which are eroded include gravels and a wide variety of sandy deposits in the Wicomico, St. Mary's, Choptank, Calvert, Naniemoy, Aquia, and Magothy Formations (Appendix A).

The mean tide range in this area varies from 0.9 and 1.2 feet. Storm surges from "annual" storms are around 2 feet, and the surges from "100-year" storms can be greater than 5-6 feet above Mean Low Water. Waves during these severe storms can be as high as 3-4 feet on top of the storm surge.

Waves in the area approach from the northeast and southeast with the longest fetches. The shallow nearshore zone is very narrow all along the Bay shoreline in Calvert County, and few reaches are sheltered from waves. However, in lower Anne Arundel County, some shorefront areas are sheltered from the longest wave fetches due to irregularities in the shoreline.

Other shorefront areas which are exposed to the longest fetches also possess broad shallow nearshore zones over which wave energies dissipate before reaching the beach. The wave and storm conditions are discussed in more detail along with other coastal processes in Chapter V.

Cases The structure case studies selected in this area include:

	Case No.	Structure
•	15	Timber bulkhead at the U.S. Naval Research Laboratories at Randle Cliff (4287 feet long).
•	16	Timber bulkhead at Dares Beach (86 feet long).
•	17	Gabions and wood groins at Scientists Cliffs.
•	18	Stone revetment at the Westinghouse Laboratories on Broad Neck, below the Bay Bridge (2100 feet long).
•	19	Gabions at Thomas Point Park in the mouth of the South River.
•	20	Aluminum bulkhead at Hillsmere Beach on the lower South River.
•	21	Concrete bulkhead on the south shore of the Severn River near Tolly Point.
•	22	Asbestos cement bulkhead on the south shore of the South River near Hillsmere Beach.
•	23	Timber bulkhead (230 feet long) at Turkey Point on the lower South River, with rubble groins.
•	24	Long Point

The following pages present brief descriptions of each structure, and nearshore bottom profiles collected at the sites. Some of the structures assigned as cases were in good condition, but some have experienced partial failure. A few of the structures evaluated do not have adequate height to prevent wave overtopping during storms, as shown on the nearshore bottom profiles. Continued erosion of the fastland was also noticeable in some cases.

The paragraphs below contain suggestions for maintenance, redesigns and improvements for several of the structures to correct serious design deficiencies.

Case 16. A timber bulkhead at Dares Beach

The timber bulkhead at this property is too low to adequately protect the upland property from splash-over and cliff base erosion. From information developed in the next chapter, a wall height of 8 ft. above the bottom would be recommended at this site to accomodate wave run-up from the "Annual Storm". This is almost 2 ft. higher than at present, and this new elevation will still not prevent bank erosion. The weathering of the cliffs will continue until a vegetated equilibrium slope is achieved. This slope could be artifically maintained by slope stability measures similar to those used by highway engineers, such as small stone and/or vegetative measures.

Case 17. Gabions and wood groins at Scientists Cliff

Scientists Cliffs (+60 feet) seem to be receding at a slow rate partly because their sediments are very durable. Cliff damage is largely the result of the erosion of the cliff base by waves and weathering. The present groin system at the base of the cliffs is not sufficient to maintain a beach which is adequately wide to provide cliff protection as well as to limit the transport of weathering products away from the area. There are several alternate means to stabilize the beach and cliffs in this area. These are described below:

Alternative #1 The Randle Cliff approach - A substantial bulkhead seaward from the base of the cliffs. This bulkhead will not permit a beach to form; however, it will prohibit shoreline retreat. The rate of cliff erosion will slow, since only weathering of the upper cliff face will continue to dislodge the fastland sediments. A groin system which would be filled with sand at the time of construction, and then periodically checked and nourished, could maintain a beach in this case.

Alternative #2 Stone revetment - This will work in the same manner as Alternative #1, but a natural beach may form in front of the revetment. Groins may be necessary to hold the beach, since there is a significant net littoral transport of sediment to the south. If a revetment is built slightly seaward from the present base of the cliffs, the initial cost of the protection will be higher, but weathering of the cliff face should be slowed somewhat due to the structural protection against wave action. This could reduce the hazard to shorefront homes; but, erosion of the exposed cliff face will still continue.

Alternative #3 <u>Groins and beachfill</u> - The existing groin field has not trapped a significant volume of sand. By ensuring that the groins are sand-tight and by using beach nourishment, a significantly-wide (say 20 ft.) beach may be constructed. This beach will serve as a buffer to storms, but will require maintenance. Otherwise, after a few years, the beach will be gone.

All the above approaches would work to forestall erosion; however, they are all expensive and would require a concerted community-wide action along the entire shoreline reach.

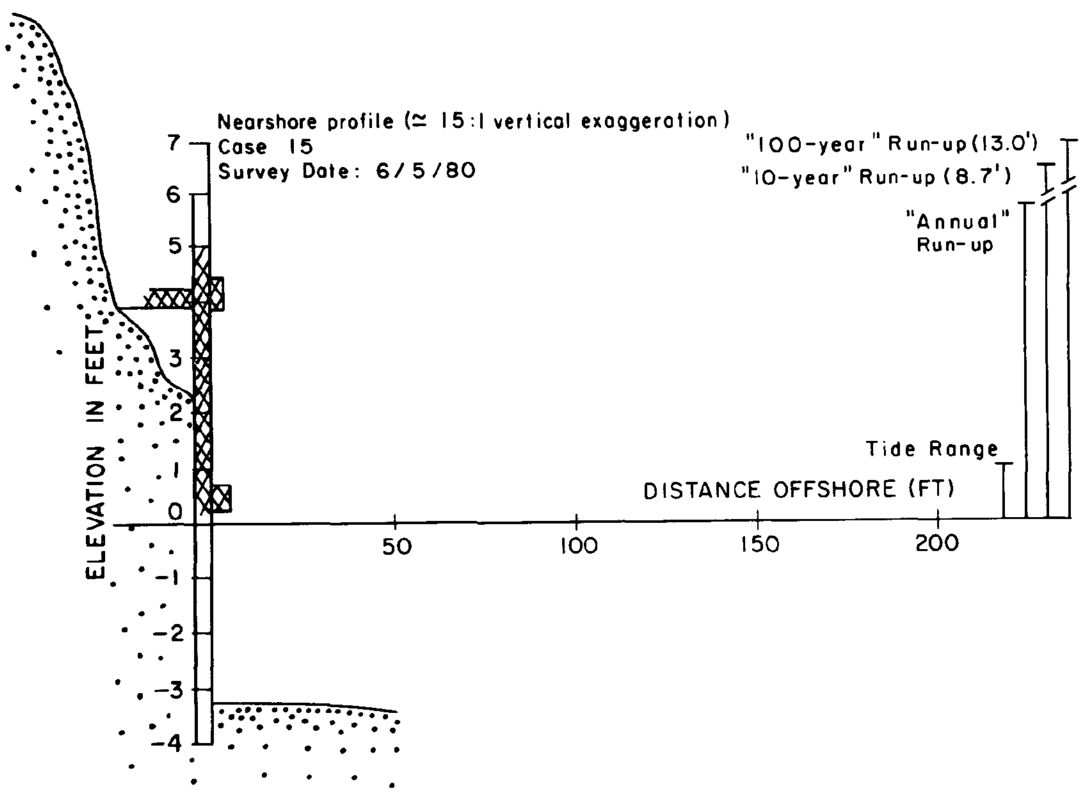
<u>Case AA-2 Gabions at Thomas Pt. Park in the mouth of South River</u>

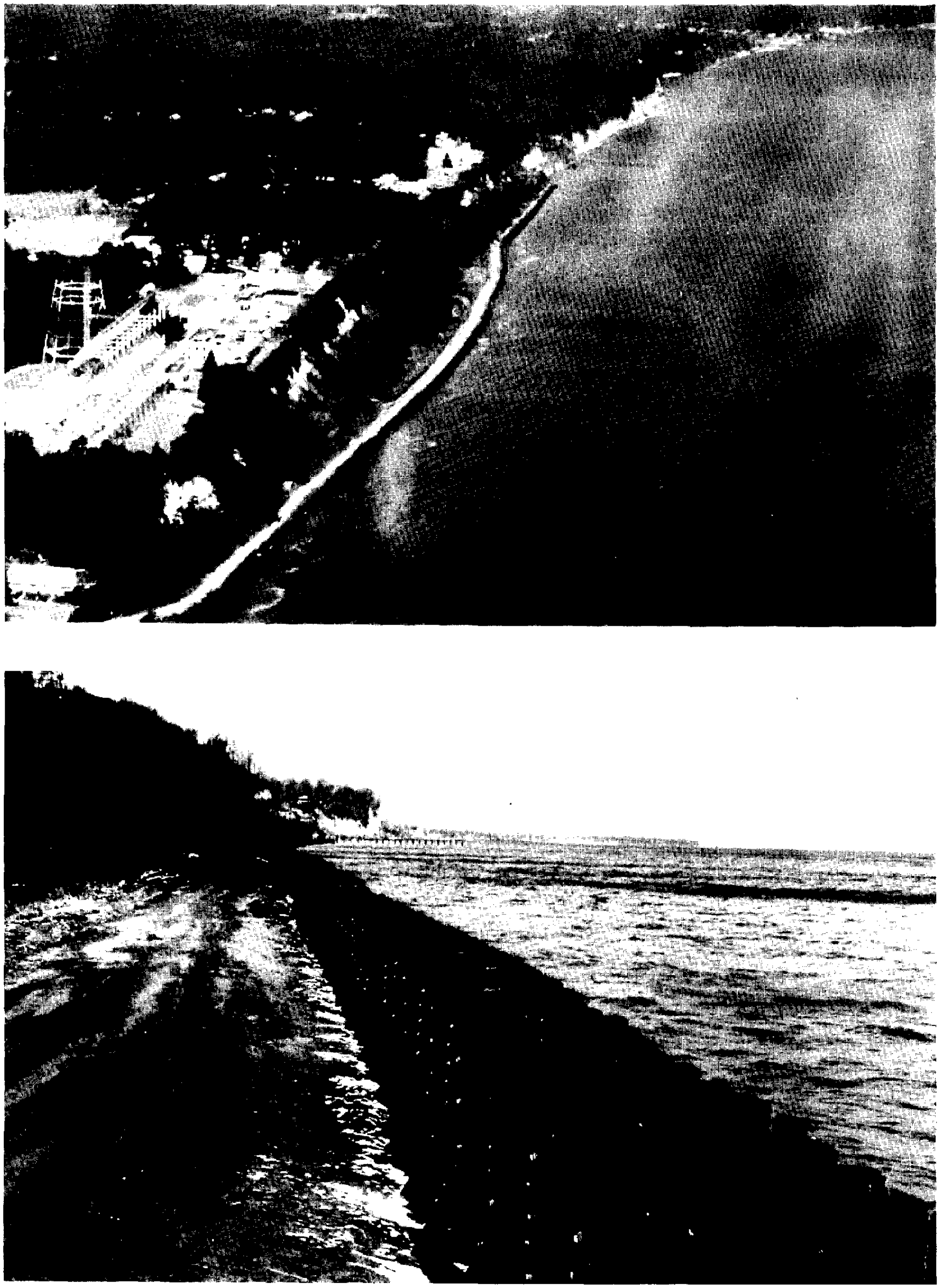
The initial design of this system appears to be adequate for wave and storm conditions. It is important, however, that a proper maintenance program be instituted. As the gabions fail and the stones wash out, the structural integrity begins to deteriorate. A replacement program for failed gabions must be instituted now and a biennial effort should be made to replace new failing gabions.

CASE 15 A TIMBER BULKHEAD AT RANDLE CLIFF

This structure was completed in 1969 at an unknown cost. The historical rate of erosion at the site was 2 ft./yr. from 1847-1934. Structure consists of creosoted sheetpile, 4 in. x 12 in. x 15 ft. long. Batter piles were installed on the seaward side, except where prohibited due to remnants of a previous 40 year-old steel sheetpile wall. The new structure was placed within 100 feet of the bluffs.

This structure is in generally good condition. Weep holes and drainage pipes were installed to keep earth pressures low behind the wall, as well as to accomodate runoff from the bluff face. Erosion of the cliff is continuing at sites alongshore where no structures are present. But at this case study site, the cliff is stabilized, as evidenced by the vegetation on the bluff face.



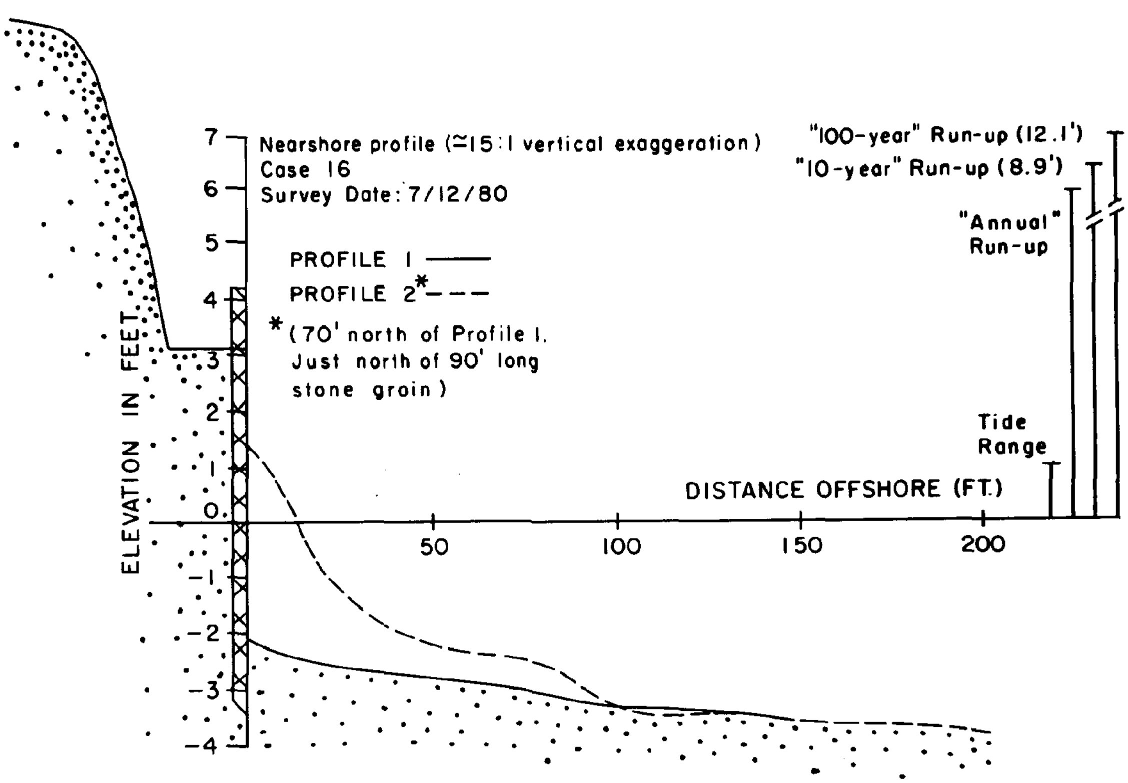


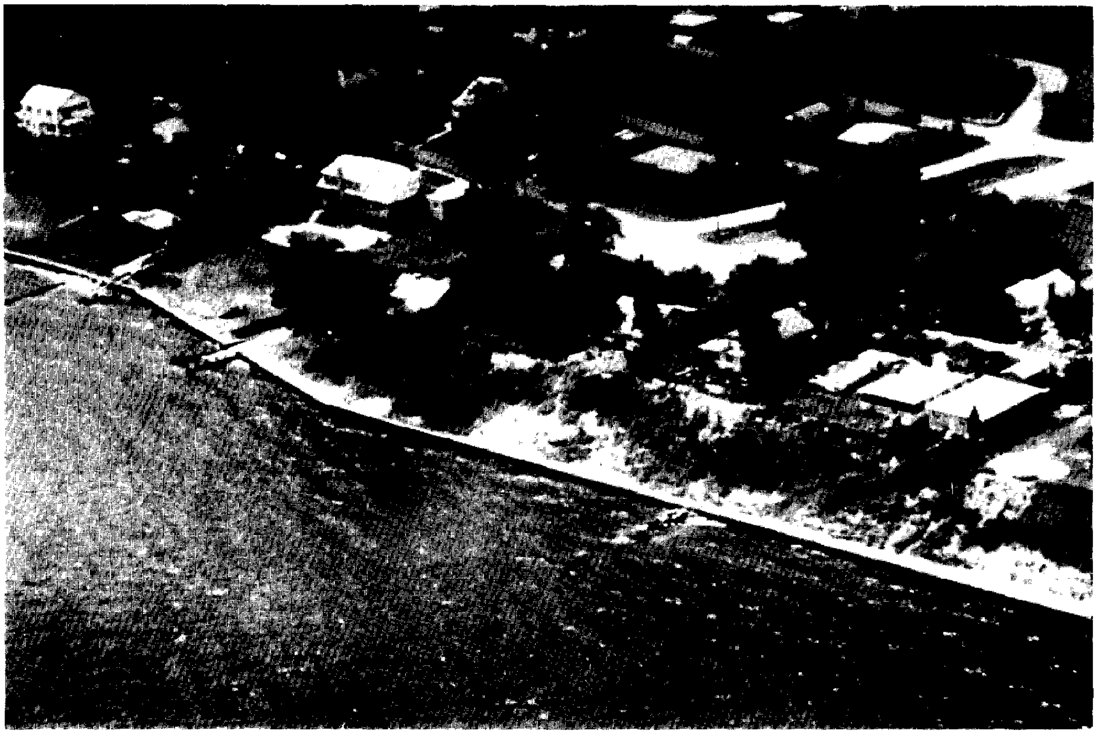
CASE 15   A TIMBER BULKHEAD AT RANDLE CLIFF

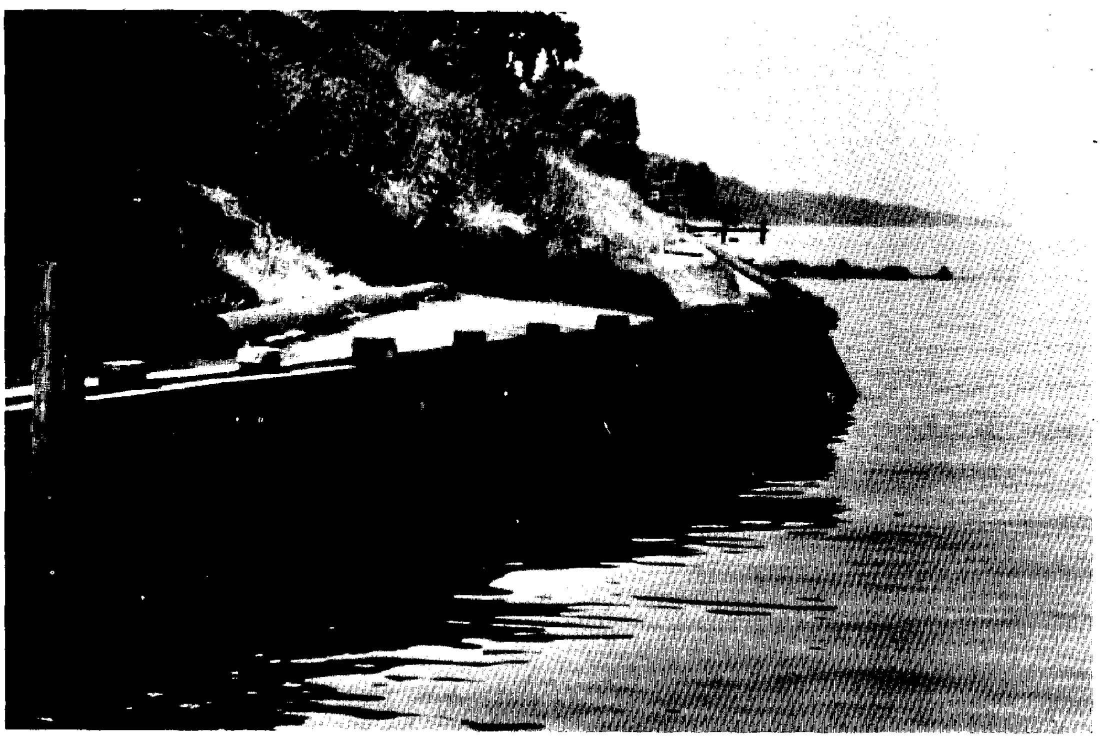
CASE 16   A TIMBER BUCKHEAD AT DARES BEACH

Structure was completed in 1975 at a cost of $122.21/ft. The historical rate of erosion at the site was 3 ft./yr. from 1848-1934. The timber bulkhead is composed of 18 ft.-long pile, and 12 ft.-long sheetpile. Several 16 ft.-long batter piles were installed on the seaward side to bolster the structure. Some stone groins are also present in the vicinity of the structure. A small amount of sand has been impounded in the fillets at the base of the groins. But elsewhere, a beach is absent.

This structure is in generally good condition. The structure extends alongshore in front of several separate properties. There is significant overwash during strong wave conditions and there is evidence of continued bluff erosion. To the south, a neighboring property has a seawall installed which is approximately 18 in . higher, and evidence of wave overtopping at that site is noticeably less severe. To the north, the case structure does not quite connect with another bulkhead, and severe erosion and removal of the bluff face has taken place in the gap.







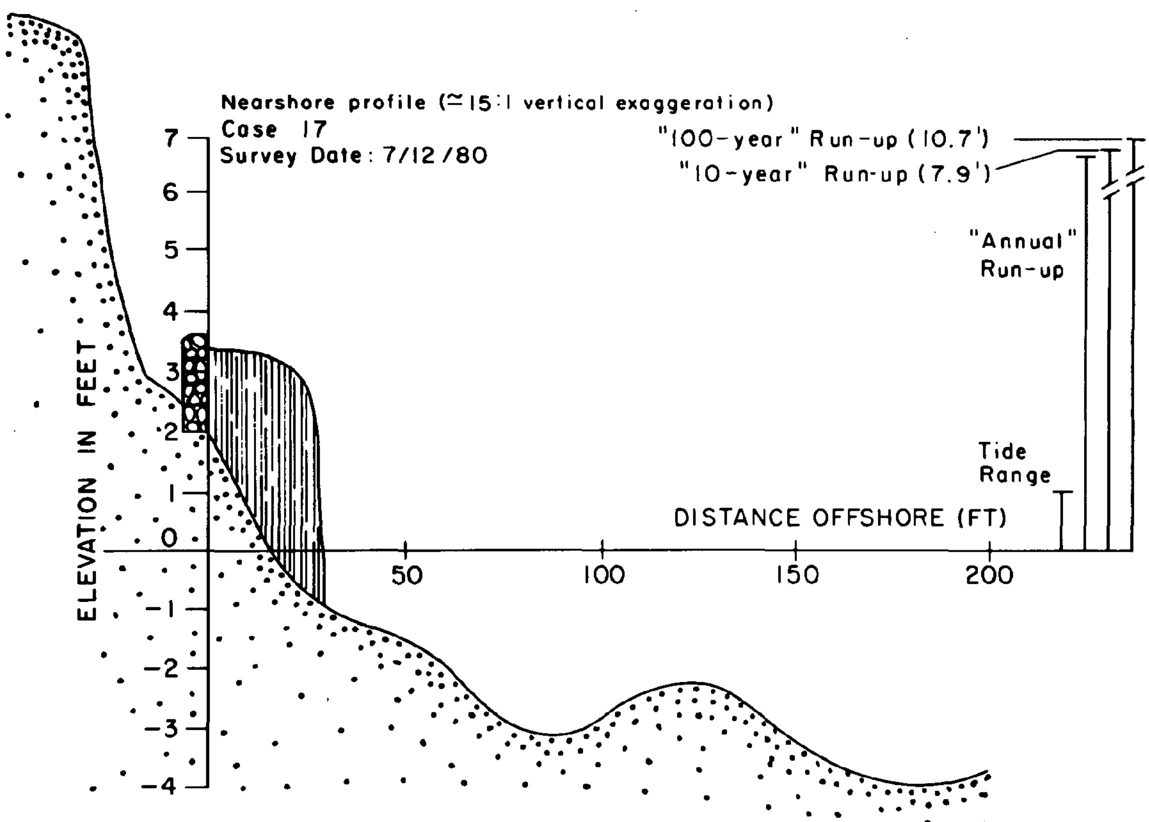
CASE 16 A TIMBER BUCKHEAD AT DARES BEACH

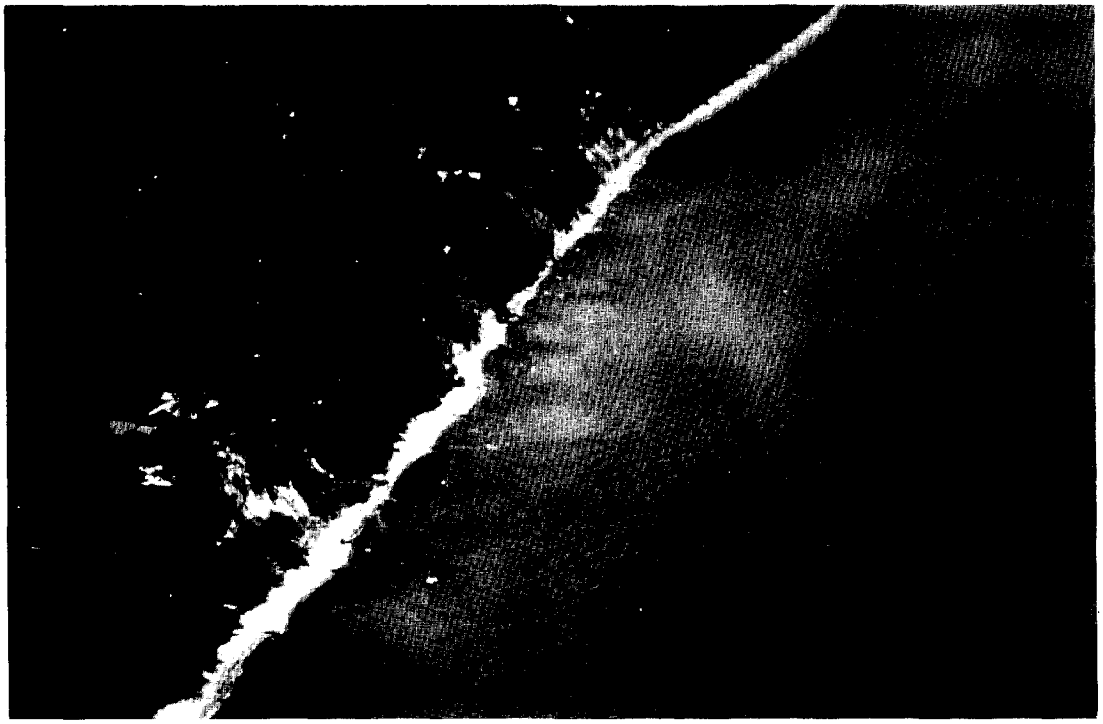
CASE 17 GABIONS AND WOOD GROINS AT SCIENTISTS CLIFFS

The historical rate of erosion at the site was about 1 ft./yr. from 1848-1943. The groins are approximately 20-30 years old. The gabions have been installed in more recent times. The cost of these structures is not known.

The structures were installed to prevent sand transport away from the base of the bluffs, and thus provide a beach to dissipate wave energy. These groins are of a unique construction. Well-rings were used to stabilize the substrate, and piles which hold the wooden groin panels in place were augered into the bottom. Later, gabions were used to extend the landward ends of the groins.

These structures are in generally fair condition. However, the groins have not stopped the wave attack on the cliffs.



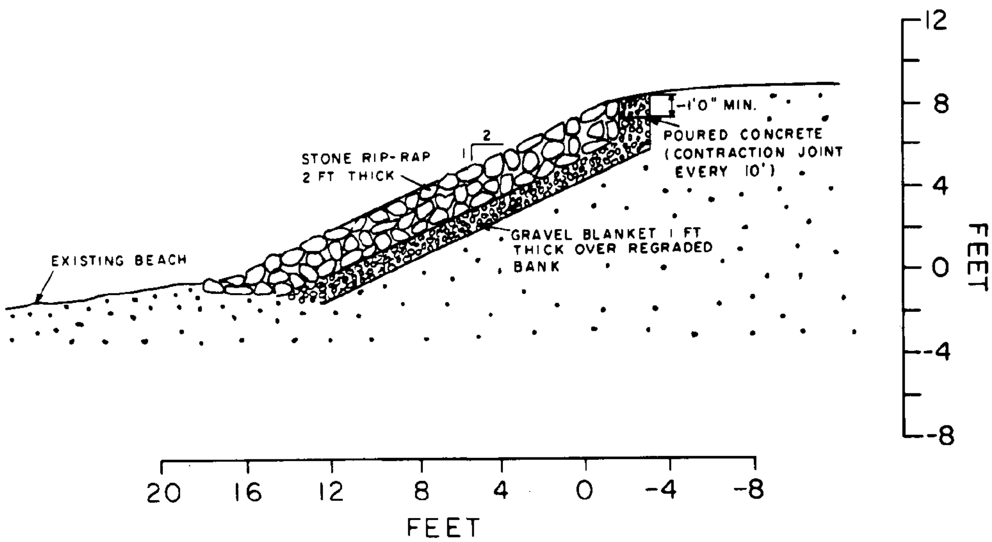


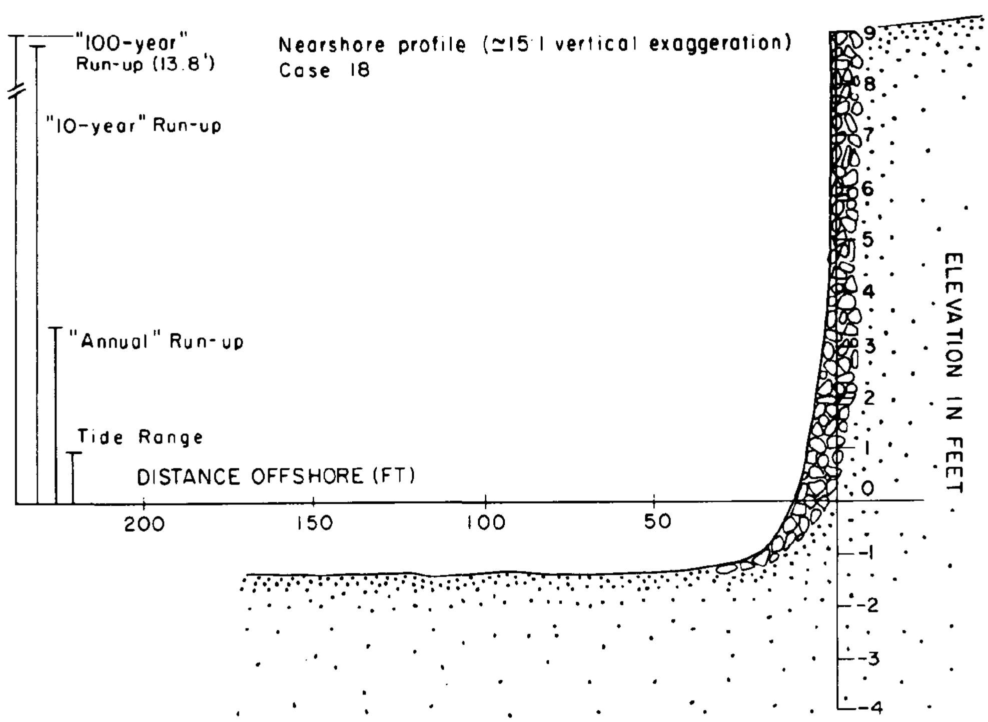
CASE 17 GABIONS AND WOOD GROINS AT SCIENTISTS CLIFFS

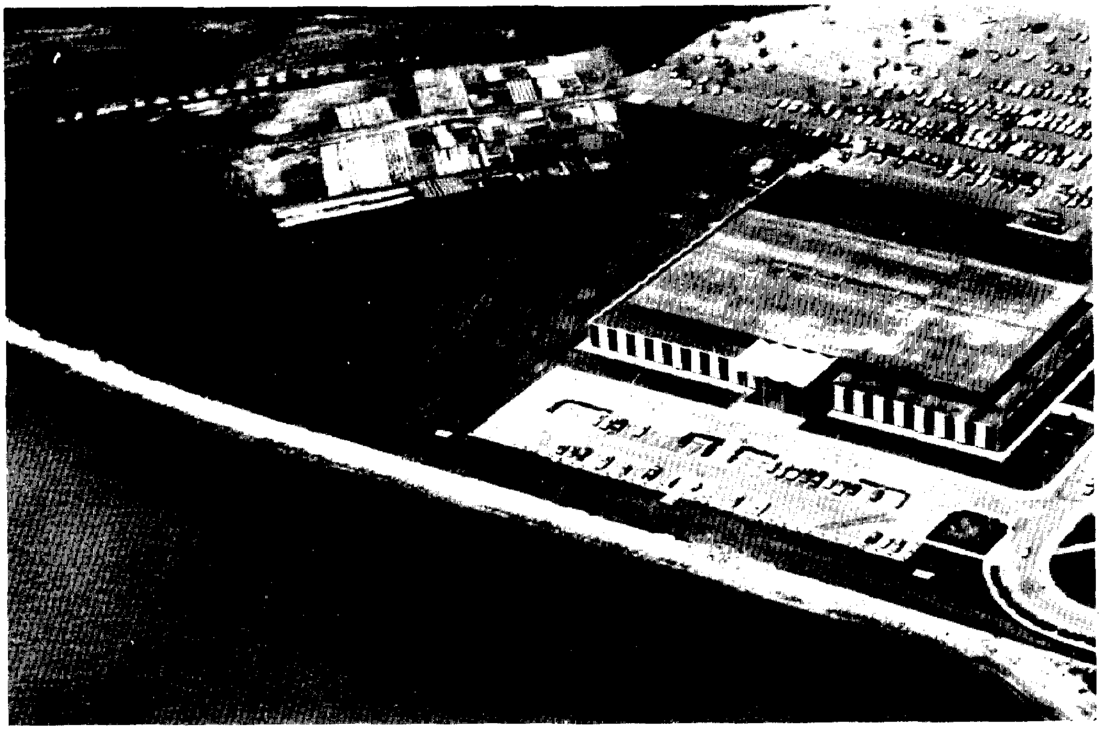
CASE 18 A STONE REVETMENT NEAR THE BAY BRIDGE

The structure was completed in 1969 at an unknown cost. The historical rate of erosion at the site was about 3 ft./yr. from 1845-1942. Revetment, on a 2:1 slope, consists of 30-300 lbs. stone in a 2 ft.-thick armor layer. A bedding layer of gravel 1 ft.-thick was placed below the armor layer. There was no filter material installed below the bedding layer. The revetment also has a concrete cap 1 ft. x 1.5 ft.-thick installed along the top.

This structure is in generally fair to poor condition. Parts of the structure failed during Tropical Storm David in early September 1979. The wall still provides some protection to the property though.





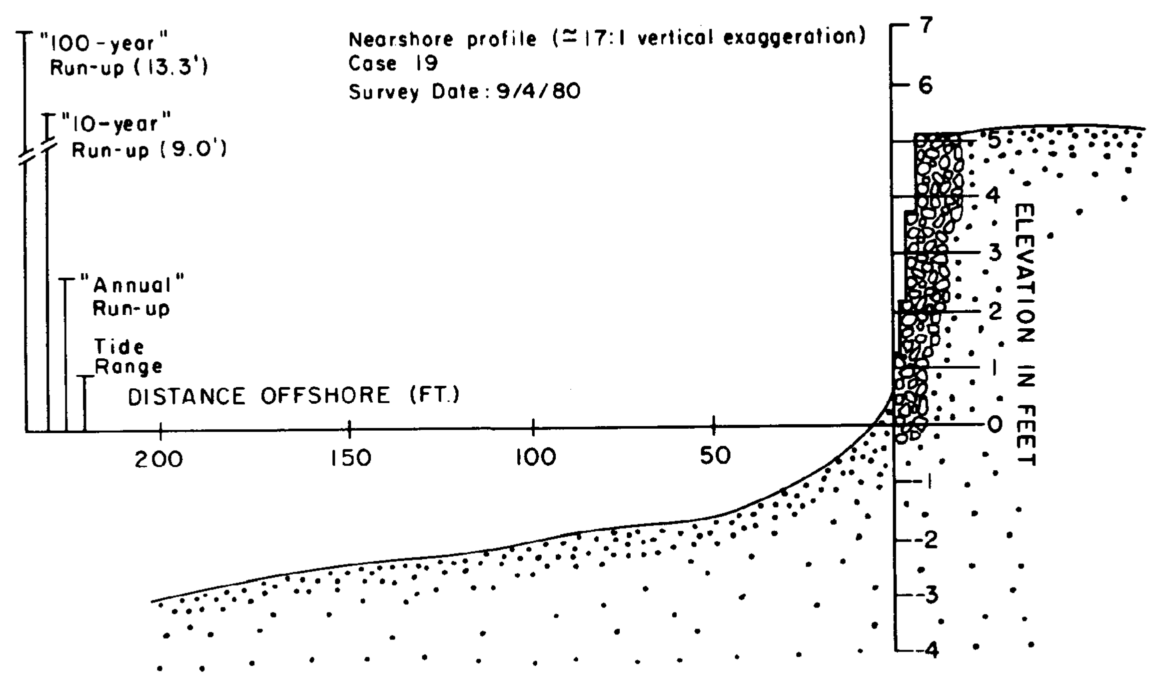


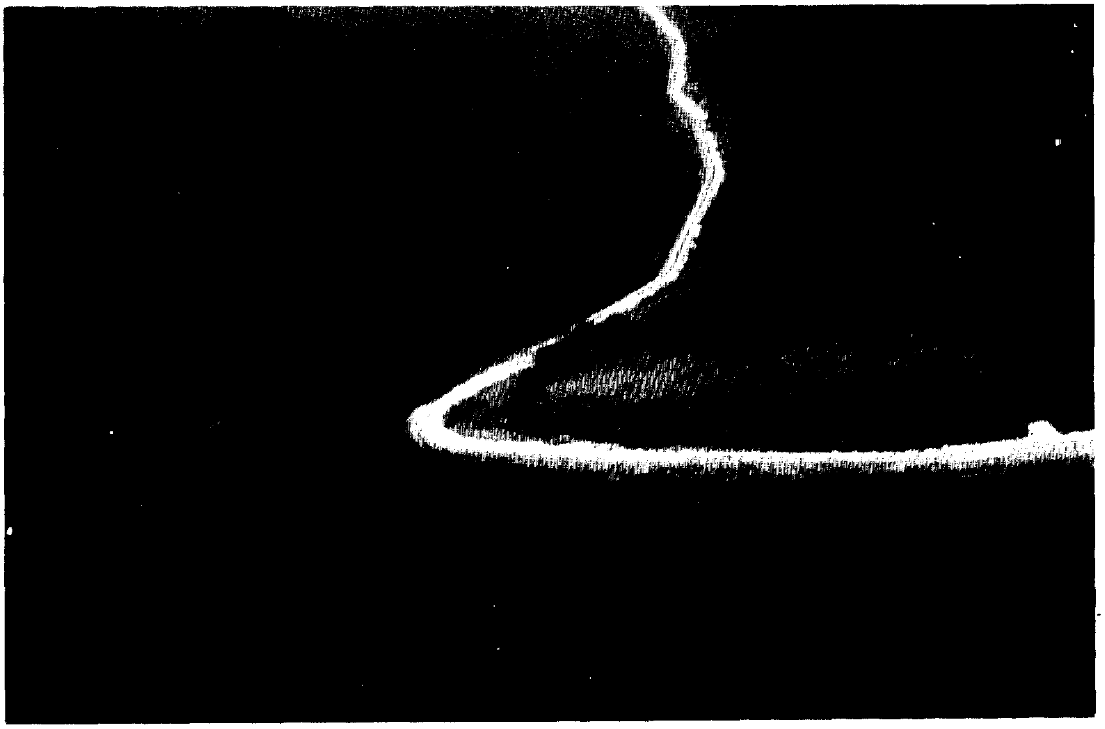
CASE 18 A STONE REVETMENT NEAR THE BAY BRIDGE

CASE 19   GABIONS AT THOMAS POINT PARK

The historical rate of erosion at the site was less than 3 ft./yr. from 1847-1970. The structure is composed of stones in plastic covered wire baskets. The gabions are stacked in 3 levels, 4 baskets-wide at the base; 2 baskets-wide in the next layer; and 1 basket on top. On the landward side, filter material was used. Behind the filter material, a 10 ft.-wide rubble apron has been installed to protect the upland from further overwash.

This structure is in generally good condition. However, a number of the gabions in the base layer are failing. The baskets corrode when the plastic coating is abraded; and, as holes form, the wave action washes out the stones. The baskets have no structural strength without the stone.



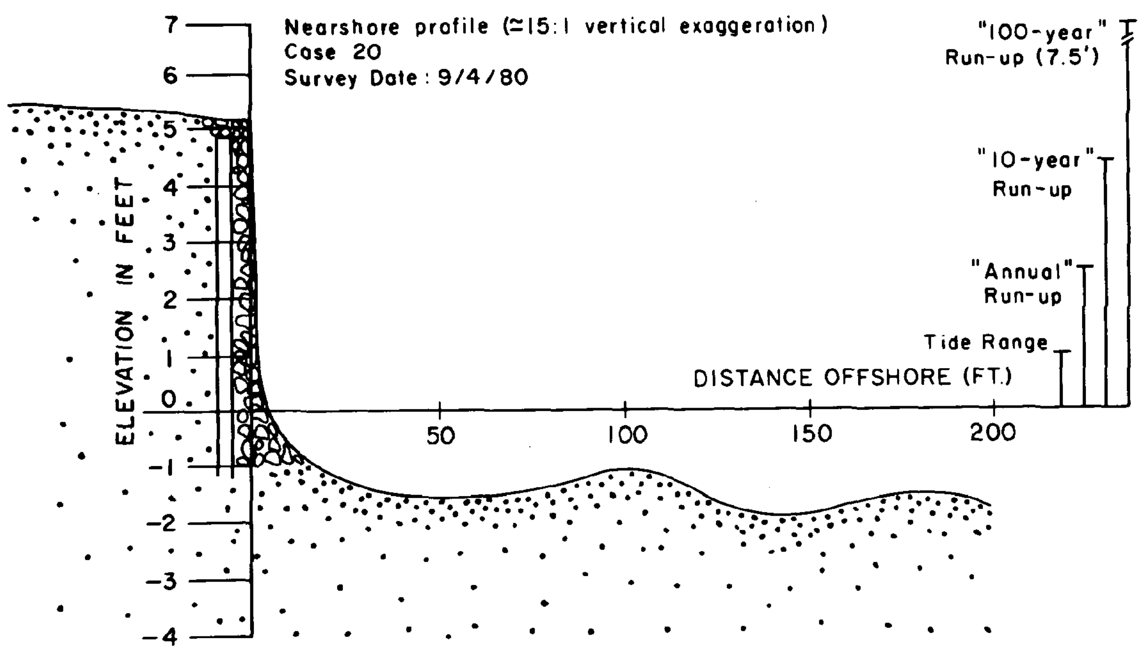


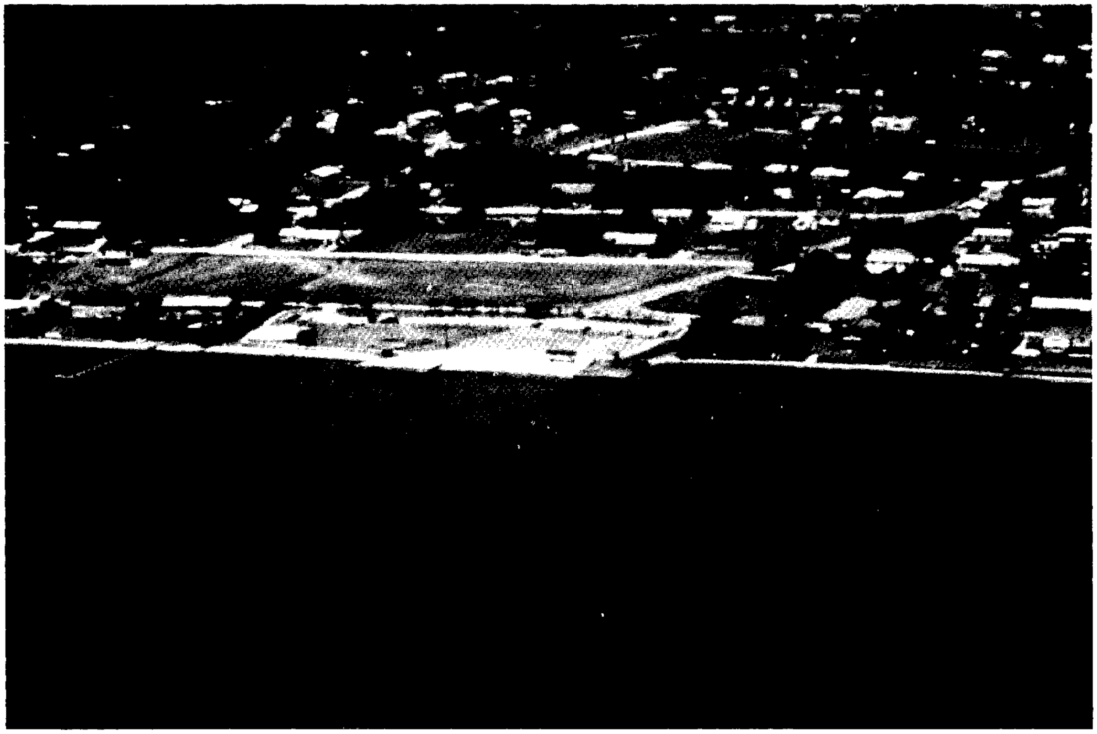
CASE 19 GABIONS AT THOMAS POINT PARK

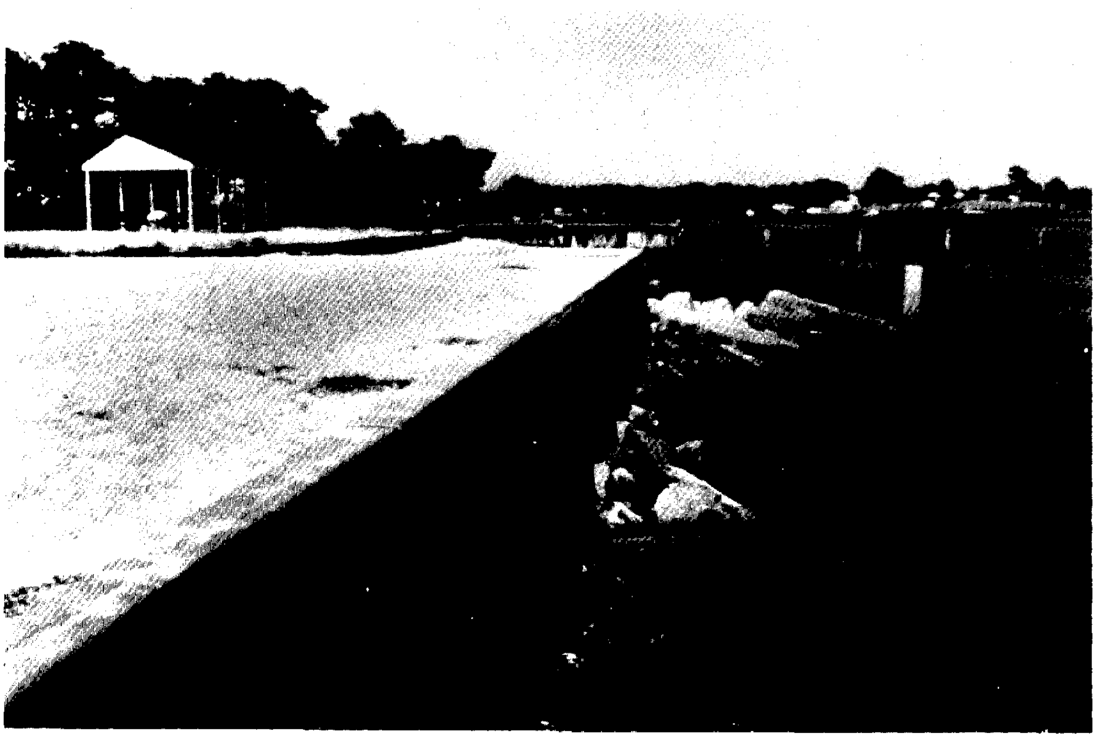
CASE 20 AN ALUMINUM BULKHEAD AT HILLSMERE SHORES

The historical rate of erosion at the site was about 1 ft./yr. from 1847-1970. The bulkheads at this site form "headlands" that extend out into the water at both ends of the beach, to prevent the beach sand from washing out. The bulkhead at one end of the beach is composed of "Shoreall" aluminum bulkheading fronted by rip-rap in an 8 ft.-wide revetment. At the other end of the park, a timber bulkhead is installed.

These structures are in generally good condition. At both structures, grass is growing offshore, and is stabilizing the nearshore bottom.



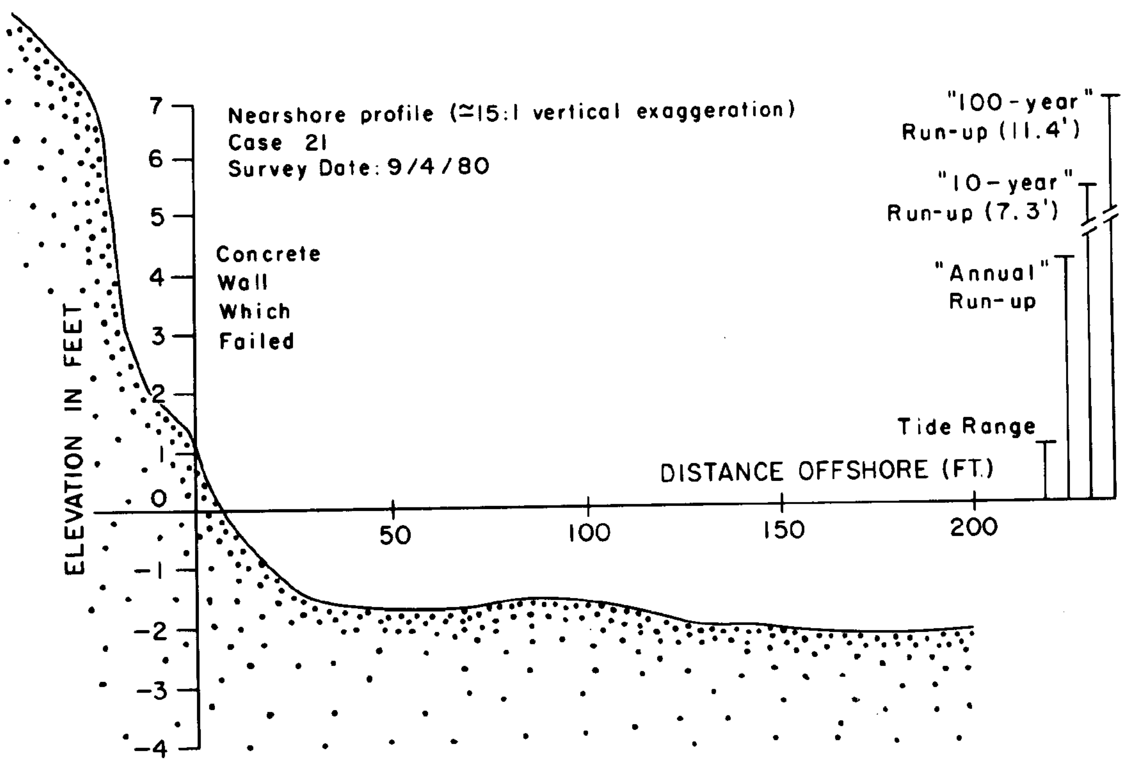


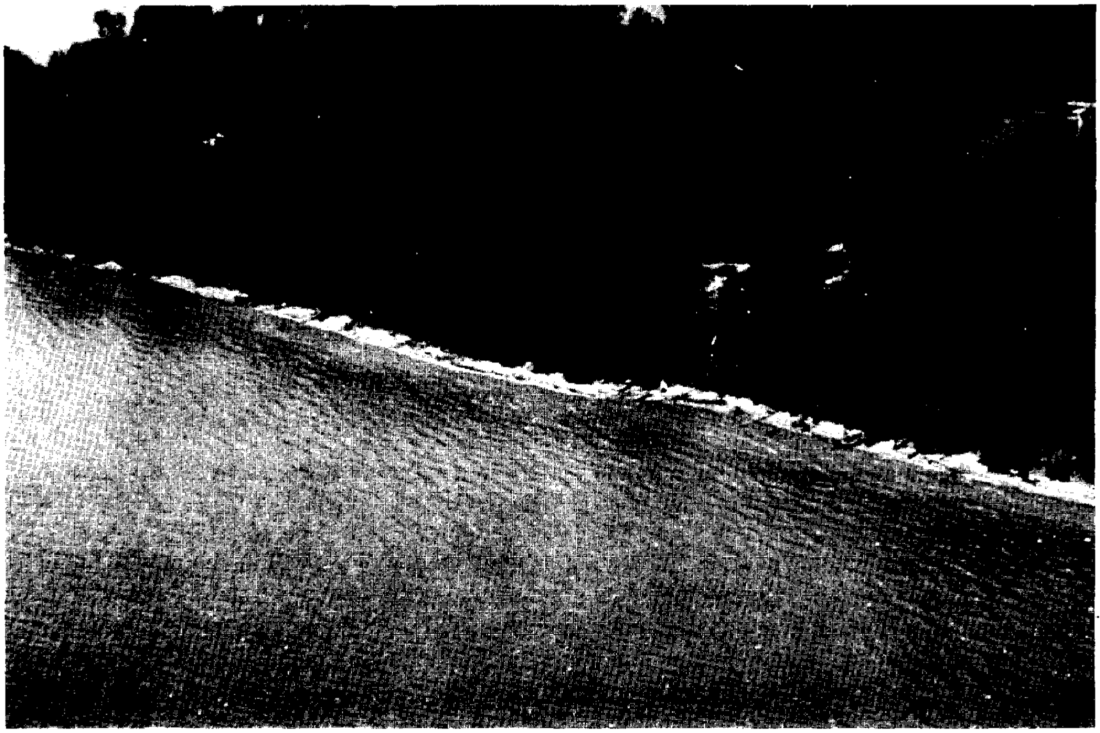


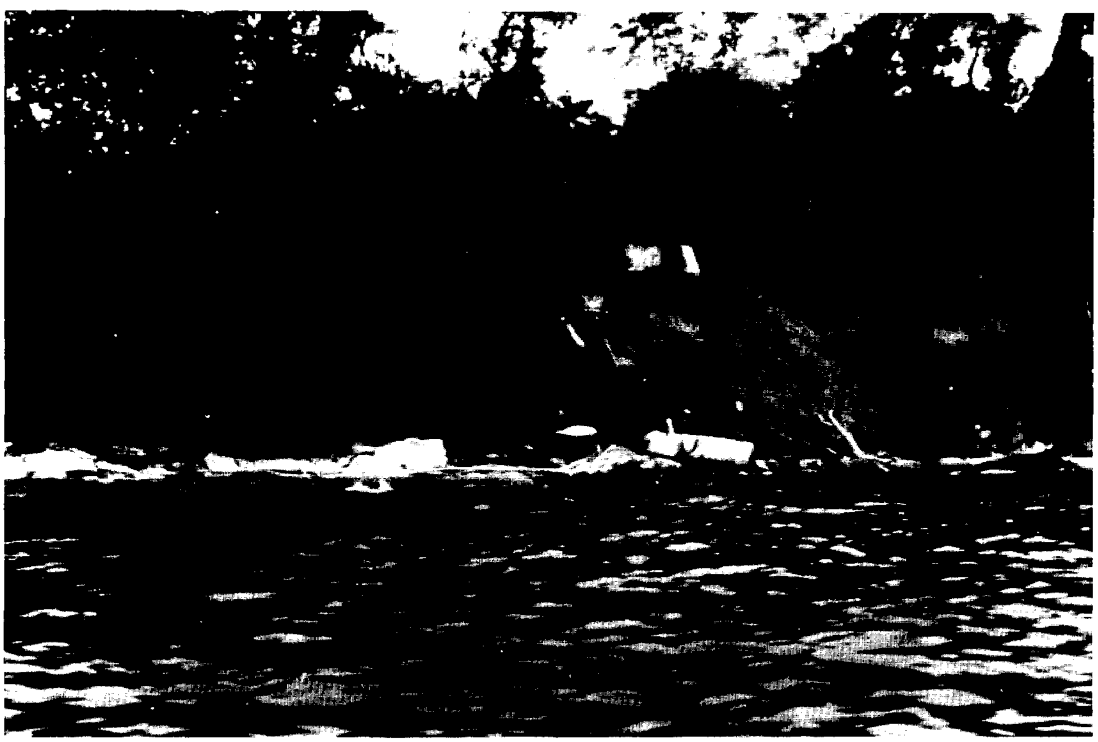
CASE 20 AN ALUMINUM BULKHEAD AT HILLSMERE SHORES

CASE 21 A CONCRETE BULKHEAD NEAR TOLLEY POINT

This structure protects a site composed of high vegetated bank with a failed patio. The date and cost of the structure are not known. Structure was composed of a concrete wall, apparently about 5 feet high, which failed several years ago, judging from the condition of the remaining rubble on the beach. Continuing erosion has resulted in the loss of a house apparently located on top of the bank.





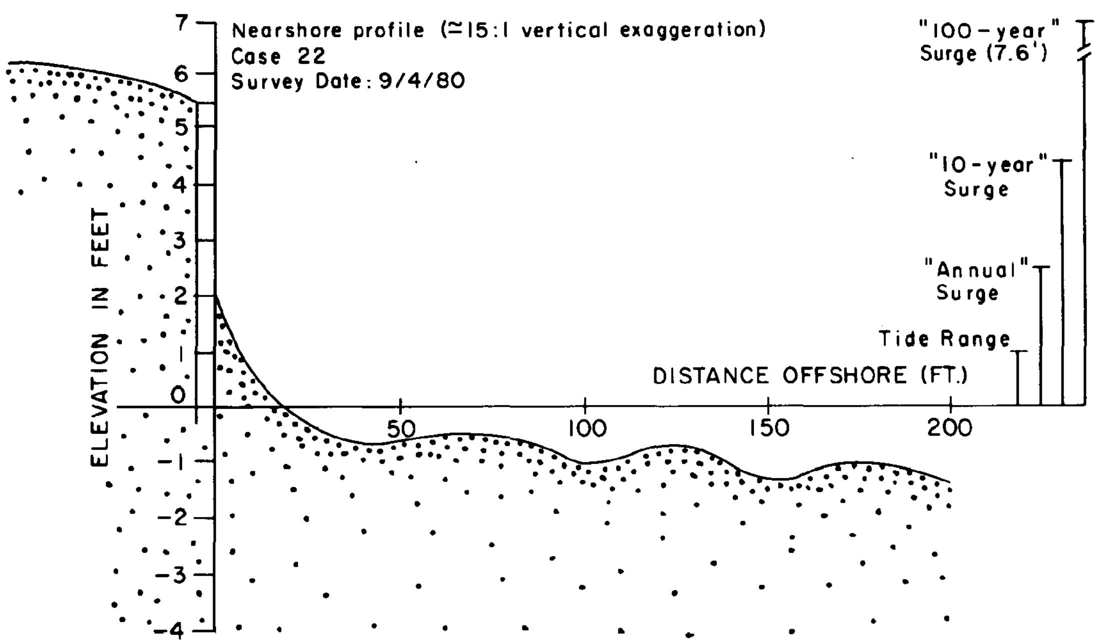


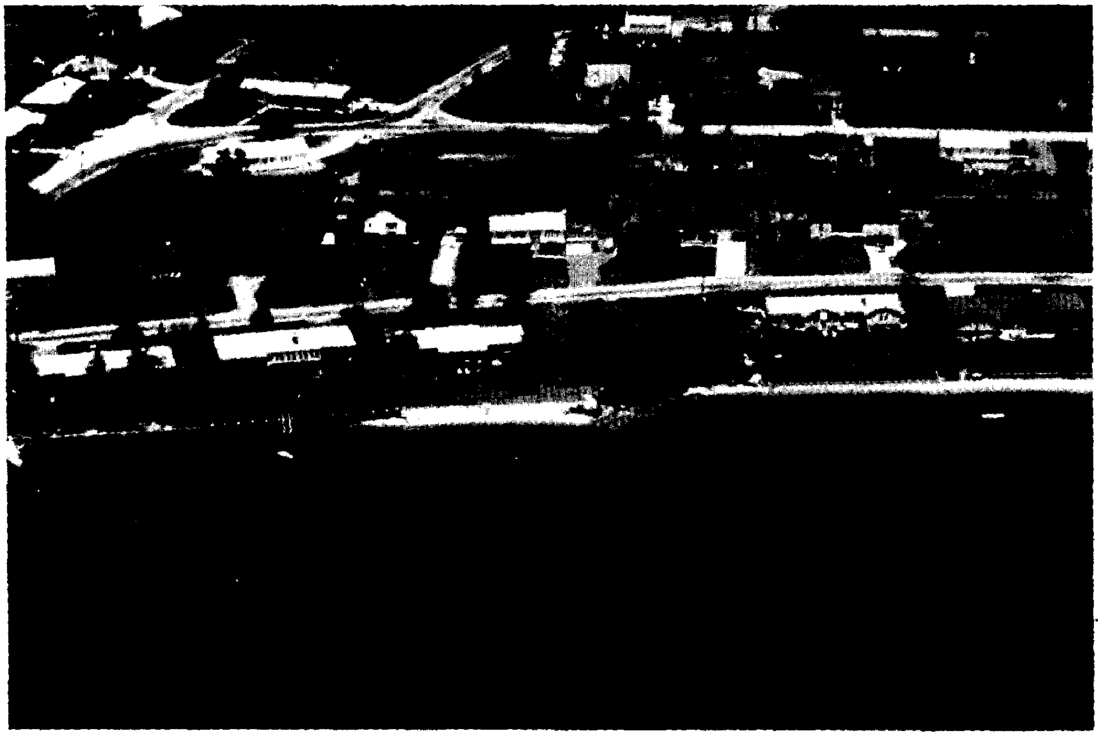
CASE 21 A CONCRETE BULKHEAD NEAR TOLLEY POINT

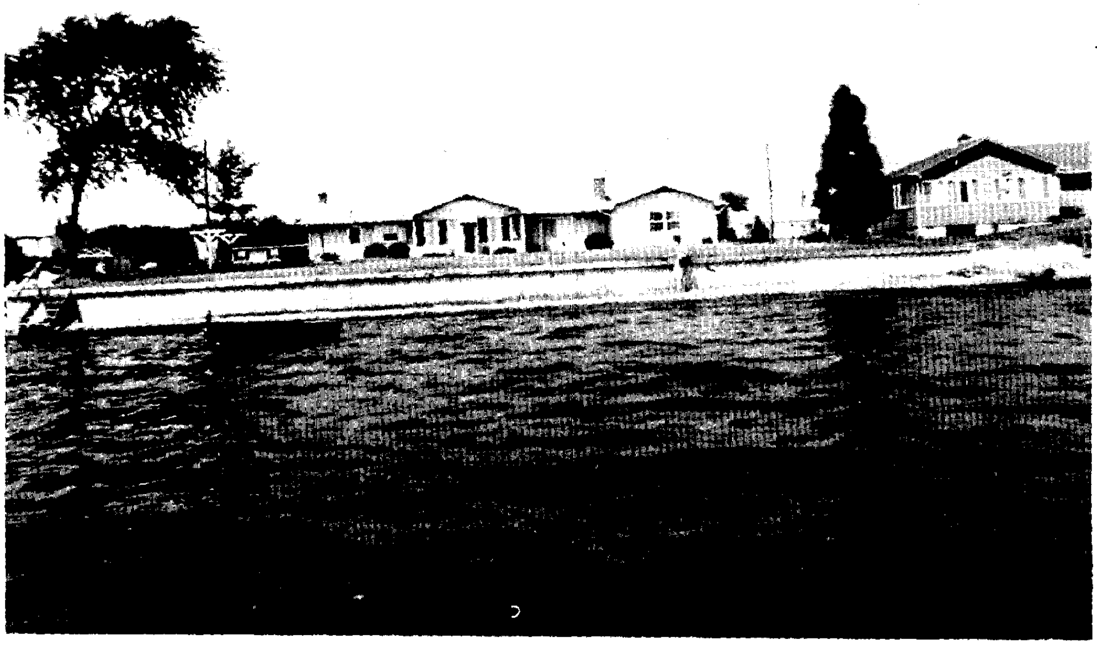
CASE 22 AN ASBESTOS CEMENT BULKHEAD NEAR HILLSMERE BEACH

This structure protects a site composed of a low grassy bank. The date and cost of the structure are not known. The structure is composed of asbestos concrete sheet pile panels. Filter materials were used behind the wall. Stone groins flank the structure at both ends alongshore. The groins are very low, and are about 65 feet long.

This structure is in generally fair to good condition. It was damaged during Tropical Storm David in early September 1979. At present, a crack extends across several of the sheet pile panels, but filter cloth behind the wall helps to keep the fastland from washing out. Although the groins are very low on the shoreline profile, there is a beach present in the vicinity of the site. Offshore is composed of very fine gray muddy sand.



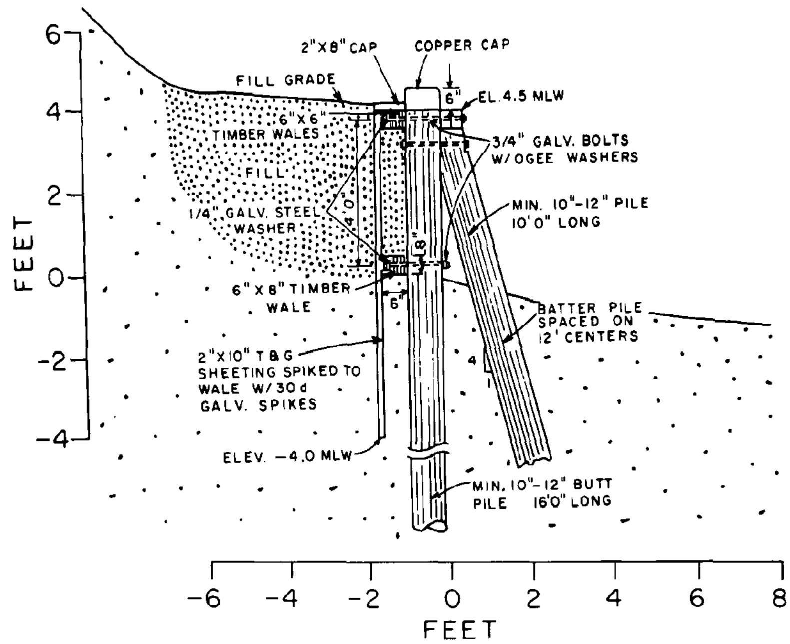


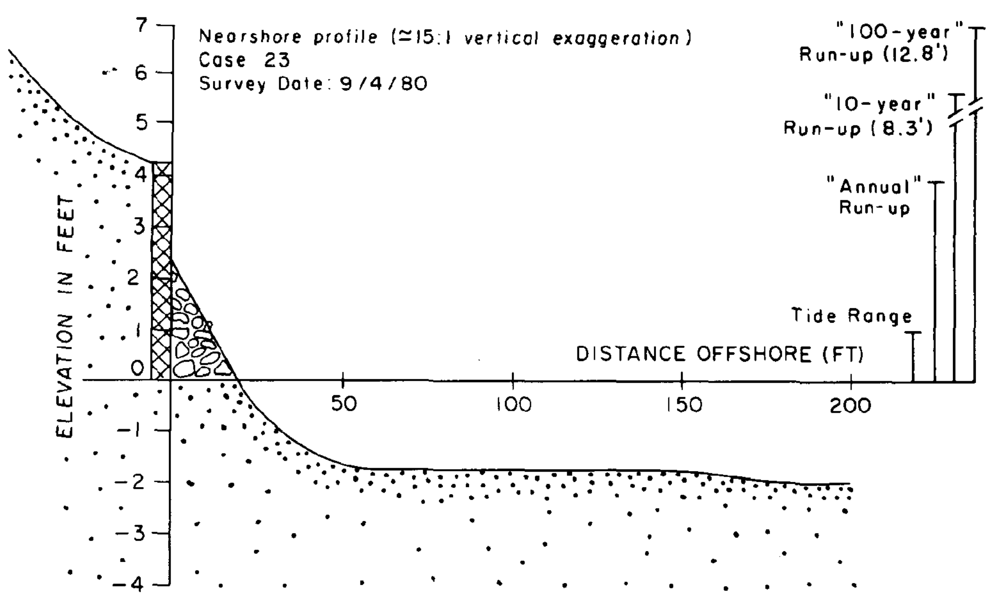


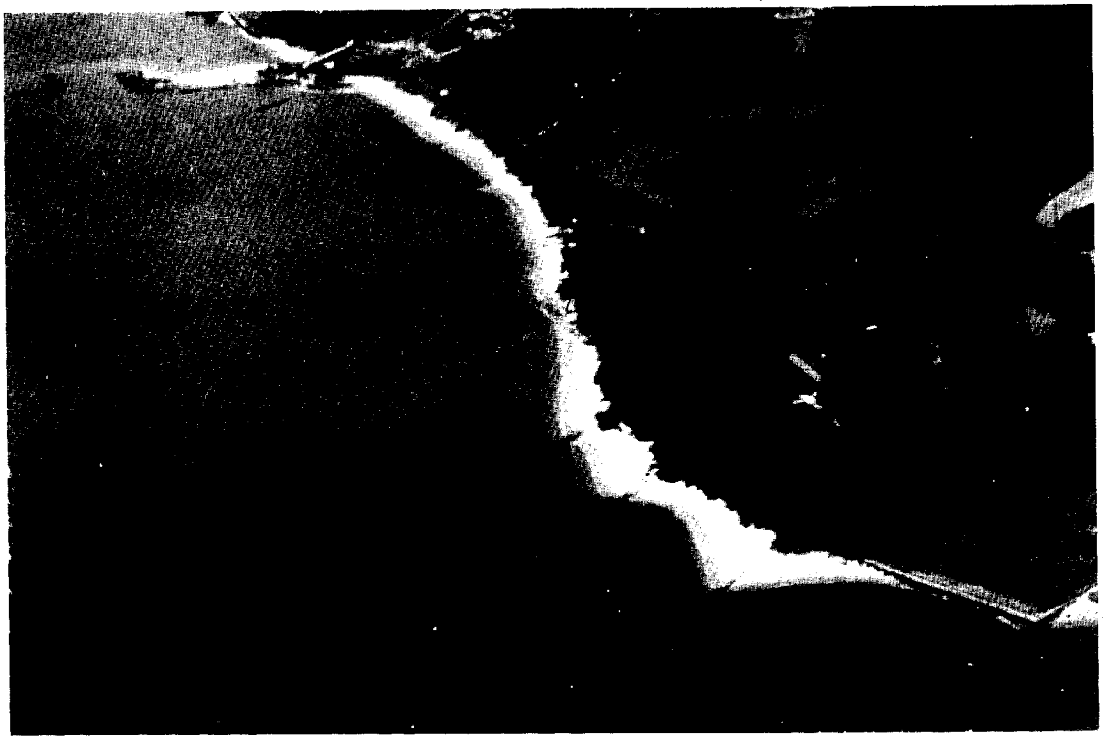
CASE 22 AN ABESTOS CEMENT BULKHEAD NEAR HILLSMERE BEACH

CASE 23 A TIMBER BULKHEAD WITH RUBBLE GROINS ON THE LOWER SOUTH RIVER

Structure was completed in 1970 at a cost of $48.70/ft. The historical rate of erosion at the site was 1.5 ft./yr. from 1847-1970. Structure consists of timber bulkhead which is angular in planform. To the south alongshore, the neighboring property has installed several 8 ft.-long concrete pipes laid out perpendicular from the shoreline to act as groins. To the north alongshore, 3 concrete rubble groins are present. The bulkhead is in generally good condition. The rubble groins are extremely effective in trapping littoral sediments, and pocket beaches are present between the groins. The concrete pipe groins have trapped some sediments to form a smaller beach, but these structures are not sand-tight.







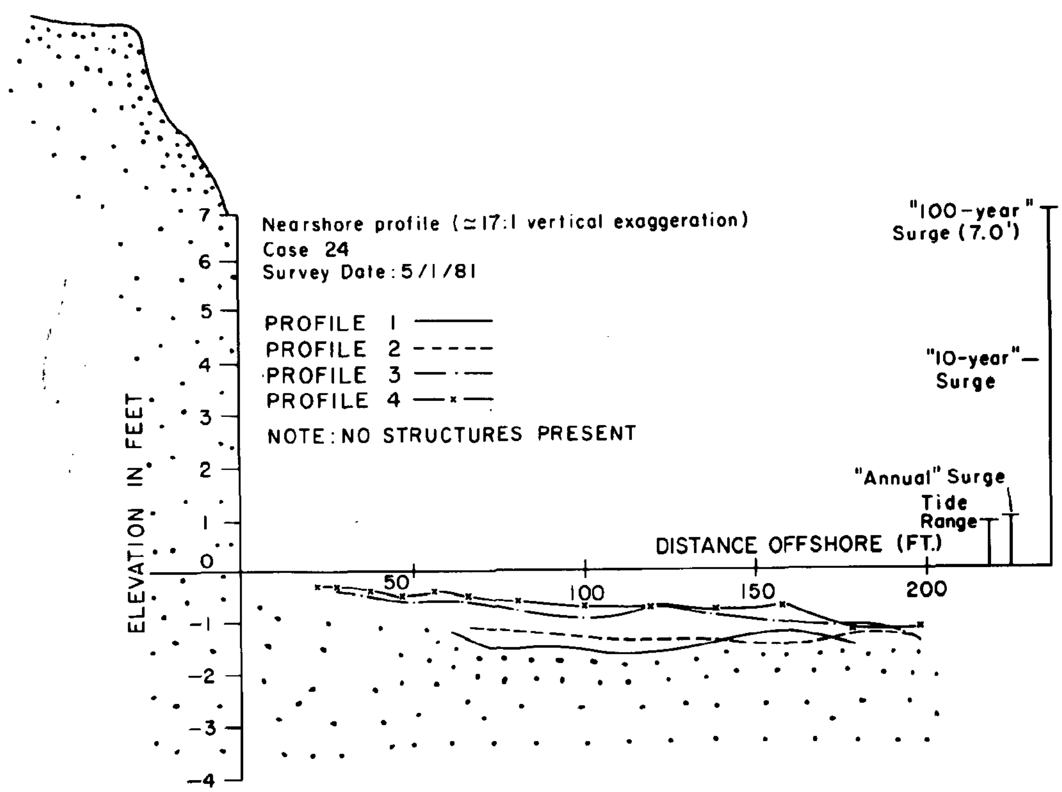
CASE 23 A TIMBER BULKHEAD WITH RUBBLE GROINS ON THE LOWER SOUTH RIVER

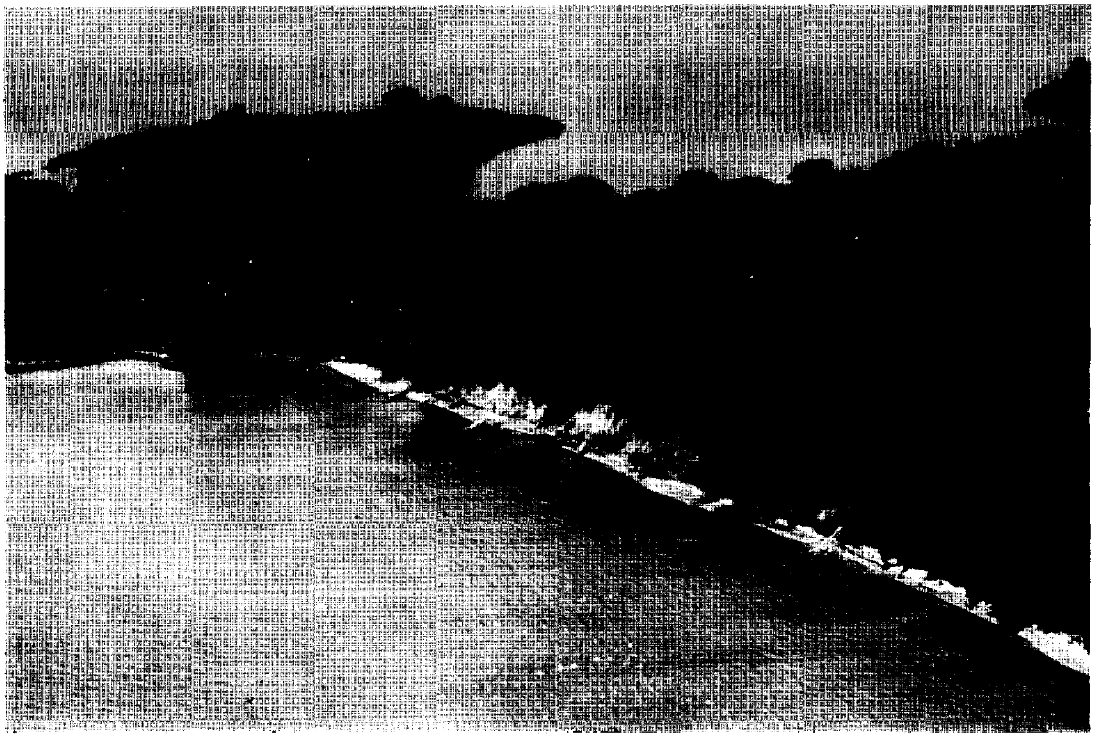
CASE 24 LONG POINT

There are no structures presently at this site, though remains of a small bulkhead are evident. The historical rate of erosion at the site was up to a maximum of 2.5 feet per year (at the middle of this shoreline reach) from 1847-1970.

Mayo Point to Long Point is a stretch of shoreline containing bluffs ranging from 15 to 30 feet in height, and a beach approximately 10 feet wide at high tide. Mayo Point appears to be accreting. However, the remaining portion of the shoreline reach is experiencing erosion.

Relics of the bulkhead structure provide evidence that the bulkheading utilized 4 inch piles with 2 inch x 12 inch, and 2 inch x 7 inch planking, 2 to 5 planks high. Presently this structure is having no beneficial effect on the shoreline.





CASE 24 LONG POINT

E. Cases on the Upper Western Shore of Chesapeake Bay

This area of the northern Chesapeake Bay shoreline (Figures 2.8 and 2.9) contains portions of Anne Arundel County, Baltimore and Harford Counties, and Cecil County. The sections below present a brief physical description of the shorelines and coastal processes, followed by a discussion of case studies which were selected from this area.

SHORELINE DESCRIPTIONS

Upper Anne Arundel County shoreline The upper Anne Arundel County shoreline extends from the Bay Bridge to Baltimore Harbor. This area is characterized by gently rolling hills between 15 and 70 feet high. Along the Patapsco River shoreline, these hills end in exposed eroding shoreline banks about 20 feet high in many spots, but farther south along Gibson Island and Broad Neck, the hillsides more often slope gently down to the water's edge, or are covered by trees and shrubs.

The beaches at the base of these hillsides are of varying widths, with considerable gravel present in some spots on the shoreline profile. At Sandy Point, the beach is separated from higher ground by a wide flat terrace containing trees, grassy areas, and parking lots of the Sandy Point State Park.

Shorefront development is concentrated at the many communities shown on the map. Houses are present both in among trees on the hillsides or in open grassy areas. Erosion control structures are present in different spots, and a beach may or may not be present along with bulkheading or revetment structures. Some houses have landscaped hillsides leading down to the water.

Baltimore County The portion of the Baltimore County shoreline included in the study extends from the mouth of the Patapsco River to the mouth of the Gunpowder River. Along the Patapsco River, shorefront areas contain concentrated industrial development and many shore-erosion control structures. Some marshes are found in protected coves and at the heads of creeks. At North Point, the shorefront contains trees and grassy slopes on the grounds of the Fort Howard Veterans Hospital. Immediately north along this shoreline reach is Black Marsh, which is fronted by a wide shallow nearshore area of sandy mud.

Hart and Miller Islands are composed of marsh and woodlands on low banks, fronted by a beach and wide sandy berm. The beach on Hart Island extends into the woodlands, and trees along the shore are dying or falling off the banks into the water. This is evidence of active erosion and shoreline retreat.

Except for Baylight Beach, directly across the inlet from Hart Island, shorefront communities are located behind Hart and Miller Islands, and farther inland along the shores of Back and Middle Rivers. Homes on these reaches are either on the tops of low banks, on hillsides which slope

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Next Pages: Figure 2.8. Shoreline along upper Anne Arundel, Baltimore, Harford and Cecil Counties

Figure 2.9. Some representative shoreline profiles collected in the summer of 1981 along the upper Western Shore of Chesapeake Bay.

Figure 2.8

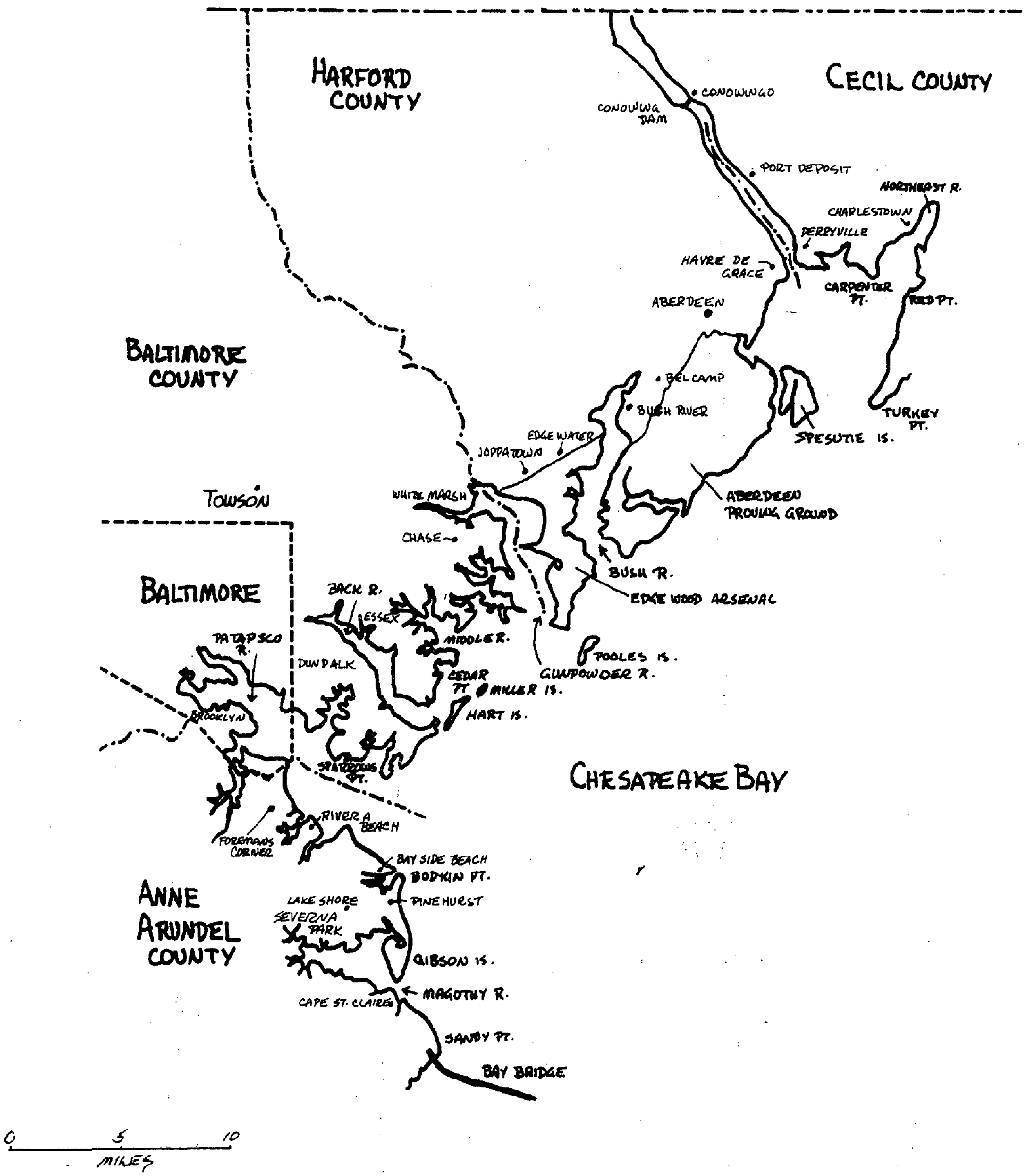
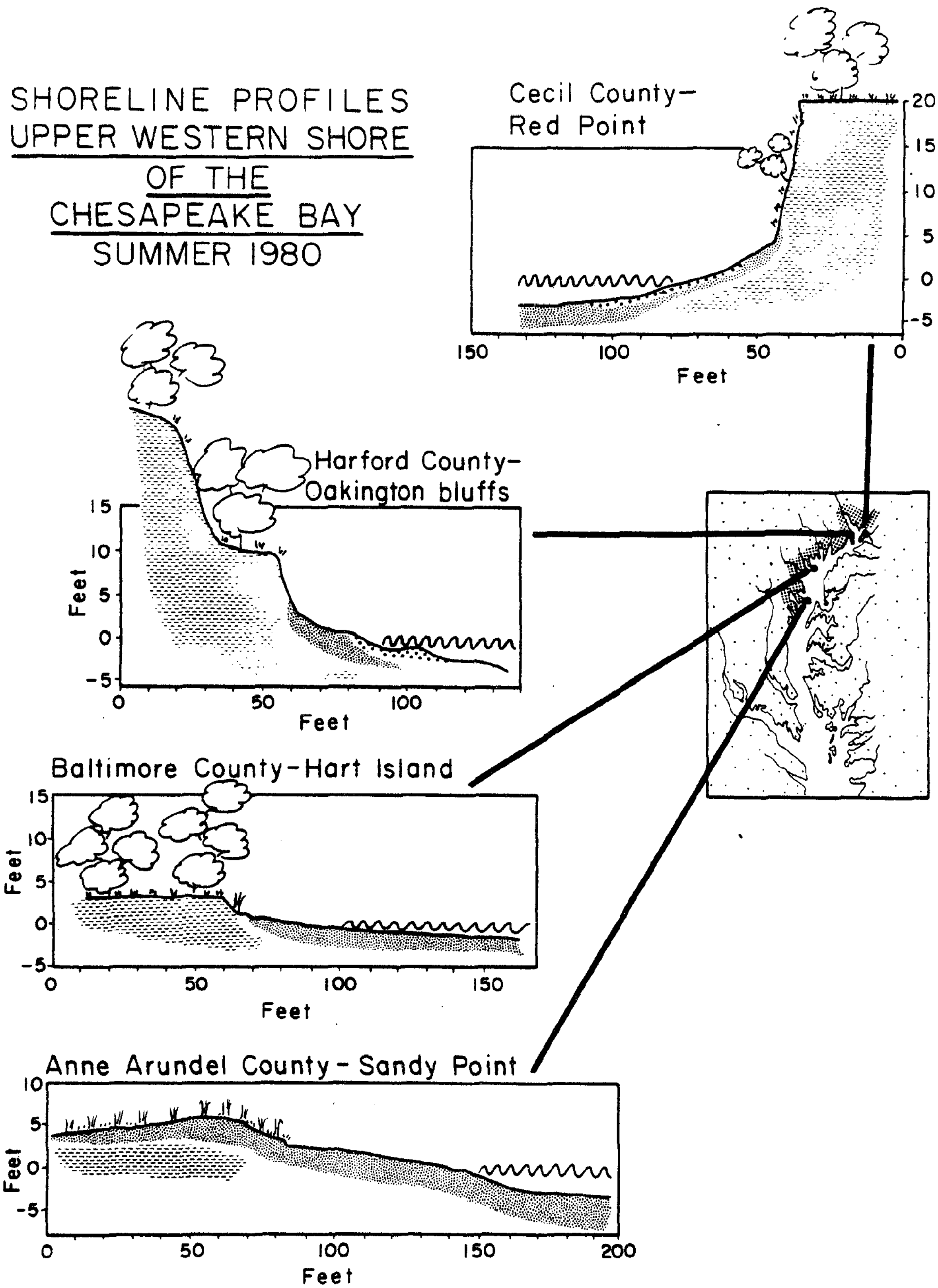

HARFORD COUNTY
CECIL COUNTY
CONOWINGO
CONOWINGA DAM
PORT DEPOSIT
NORTHEAST R.
CHARLESTOWN
PERRYVILLE
HAVRE DE GRACE
CARPENTER PT.
RED PT.
ABERDEEN
BALTIMORE COUNTY
BEL CAMP
BUSH RIVER
TURKEY PT.
SPESUTIE IS.
EDGE WATER
JOPPATOWN
WHITE MARSH
TOWSON
ABERDEEN PROVING GROUND
CHASE
BUSH R.
BALTIMORE
BACK R.
ESSEX
EDGE WOOD ARSENAL
MIDDLE R.
PATAPSCO R.
POOLES IS.
DUNDALK
CEDAR PT
MILLER IS.
GUNPOWDER R.
HART IS.
BROOKLYN
CHESAPEAKE BAY
RIVERA BEACH
FOREMANS CORNER
BAY SIDE BEACH
BODKIN PT.
ANNE ARUNDEL COUNTY
LAKE SHORE
PINEHURST
SEVERNA PARK
GIBSON IS.
CAPE ST. CLAIRE
MAGOTHY R.
SANDY PT.
BAY BRIDGE
0
5
10
MILES

Figure 2.9



SHORELINE PROFILES
UPPER WESTERN SHORE
OF THE
CHESAPEAKE BAY
SUMMER 1980
Cecil County-
Red Point
20
15
10
5
0
-5
Feet
150
100
50
0
Feet
Harford County-
Oakington bluffs
15
10
5
0
-5
Feet
0
50
100
Feet
Baltimore County-Hart Island
15
10
5
0
-5
Feet
0
50
100
150
Feet
Anne Arundel County - Sandy Point
10
5
0
-5
Feet
0
50
100
150
200
Feet

gently down to the water, or in low-lying areas protected by networks of erosion control structures.

Above Middle River, the Baltimore County shoreline contains few homes or other types of buildings, and shorefront areas consist principally of marsh interrupted by beaches which extend for 500-2000 feet along the shore. At the mouth of the Gunpowder River, the beaches on Carroll Island are very wide and are separated from the marsh by vegetated dunes.

Upstream on the Gunpowder River, exposed eroding banks again appear along the water. These contain woodlands, and the open grassy areas of Gunpowder State Park. Farther upstream, residential development is located along the shoreline overlooking the Gunpowder River delta.

Harford County The Chesapeake Bay shoreline in Harford County is largely within the boundaries of the Aberdeen Proving Ground (Edgewood Arsenal) to which public access is restricted. This shoreline contains low exposed eroding banks with beaches of varying width. Marshes are present in sheltered areas, and on isolated points of land. Shorefront areas overlooking the Bay contain woodlands and open grassy areas, but few buildings.

There are broad shallow areas upstream in the Bush River estuary which are often exposed at times of low tide. Shorefront residential development is present along the upper reaches of the Bush River, and erosion-control structures are stabilizing the low banks in some spots.

Between Aberdeen and the Town of Havre de Grace, the shoreline consists of high vegetated banks or hillsides fronted by beaches containing some gravel and cobbles. Below Havre de Grace, most shorefront areas are heavily wooded and a few houses are visible on the hillsides in among the trees. Concentrated residential and commercial development is present along with many erosion control structures on the shoreline at Havre de Grace. Farther upstream on the Susquehanna River, the shoreline contains wooded areas and exposed rock walls which are fronted by narrow sandy beaches in most spots.

Cecil County Along the Susquehanna River above the Town of Perryville, the shoreline consists of heavily-wooded land and rock walls. Large boulders and coarse gravels are the most common type of beach material on this shoreline reach. Some concentrated shorefront development protected by erosion-control structures is present at the Town of Port Deposit.

From Perryville to Charlestown, shorefront areas contain municipal development, recreational facilities, and homes, on banks ranging from 5 to 20 feet in height. The upper reaches of the Northeast River contain marsh and broad tidal flats, backed by fields and woodlands.

At the Town of Northeast, the shoreline consists of gently rolling hills ranging from 20 to 40 feet in height. In some spots, the hillsides end along the shore in exposed eroding banks, and in other places the land slopes gently down to the water's edge. Homes are present all along the shore from the Town of Northeast to Red Point. Beaches along this shoreline reach are widest in protected coves. Elsewhere, the hillsides have small beaches or erosion-control structures at their bases.

Below Red Point, the shoreline consists of high wooded hills which are protected by wide gravel beaches. A few homes and recreational facilities are present along the shore, but much of the area is within the boundary of Elk Neck State Park. The hills form a series of headlands which extend out from the land into the Northeast River. Between the headlands are a series of sheltered embayments which contain marshes or wide gravelly beaches. The bluffs bordering the shoreline are covered with trees, vines, and shrubs; but, steep eroding bluff faces are exposed in some areas, particularly along lower Elk Neck around Turkey Point. Homes and vacation cottages are present both along the bluffs and in low-lying areas next to the beach in the sheltered embayments.

Coastal Processes Many of the historical erosion rates for the upper western shore are less than 4 feet per year. But some exposed points of land have historic rates of erosion up to 8 feet per year. The shoreline sediments which are eroded include fine-grained silts, clay beds, sand, and gravel deposits of the Potomac Group, Upland Deposits, and other geologic formations listed in Appendix A.

The mean tide range in the area varies from 0.9 to 1.6 feet depending on the shoreline location. The storm surges from "annual" storms are between 1 and 2 feet, and the surges from "100-year" storms can be greater than 8 feet above Mean Low Water. Waves during these severe storms can be as high as 4 feet on top of the storm surge.

Waves in the area approach from the southeast and northeast with the longest fetches. Portions of the shoreline along the lower reaches of rivers also receive wave energy from the northwest winds blowing down the river channels. Shallow offshore areas range in width with different shoreline orientations, and much of the wave energy can dissipate before reaching the beach. The presence of substantial gravel and boulder deposits on the beach profile in some locations also helps to armor the beach against wave erosion.

The wave and storm conditions are discussed in greater detail along with the other coastal processes in Chapter V.

Cases The structure case studies selected for this area include:

Case No.	Structure
25	A buttressed concrete seawall, bulkhead, and timber groins north of Gibson Island.
26	A timber bulkhead/stone revetment on Middle River (381 feet long).
27	Gabions on Gibson Island, with groins.
28	Aluminum bulkhead on Back River (175 feet long)
29	Concrete bulkhead north of Gibson Island (approx. 2600 feet long).
30	Concrete bulkhead and concrete groins near Hawkins Point.
31	A well-ring bulkhead on Broad Neck.

The following pages present a brief description of each structure and nearshore bottom profiles collected at the sites.

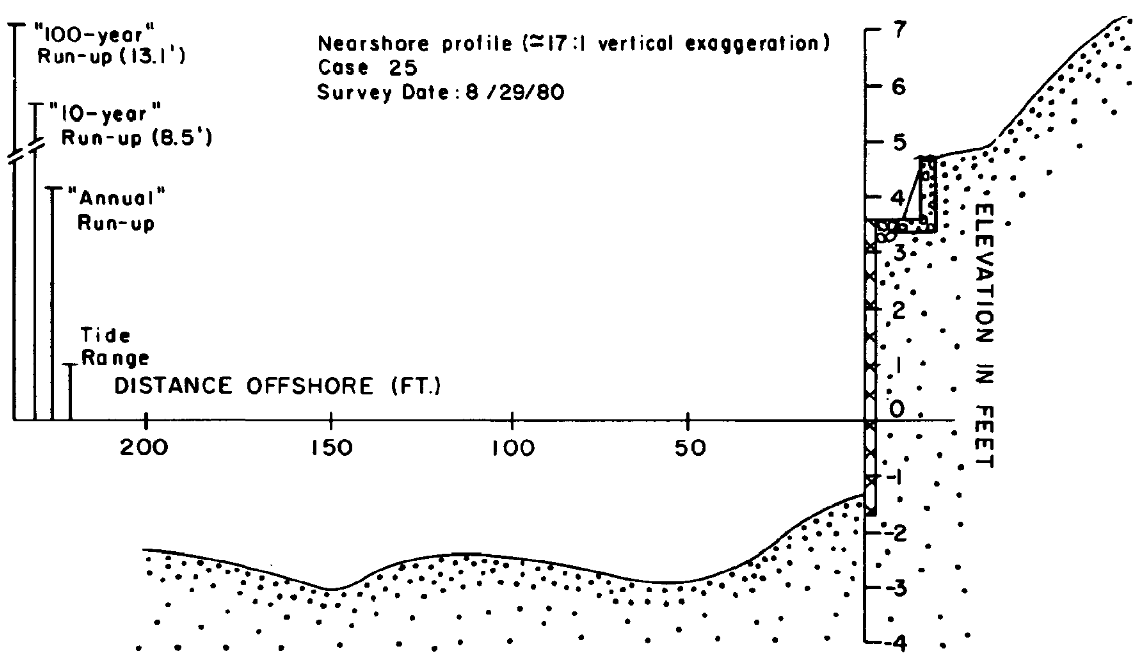
Many of the case structures appeared to be in good condition and will protect the fastland against adverse wave conditions. There were no serious problems which could not be prevented with routine maintenance. There were no redesigns suggested for this area.

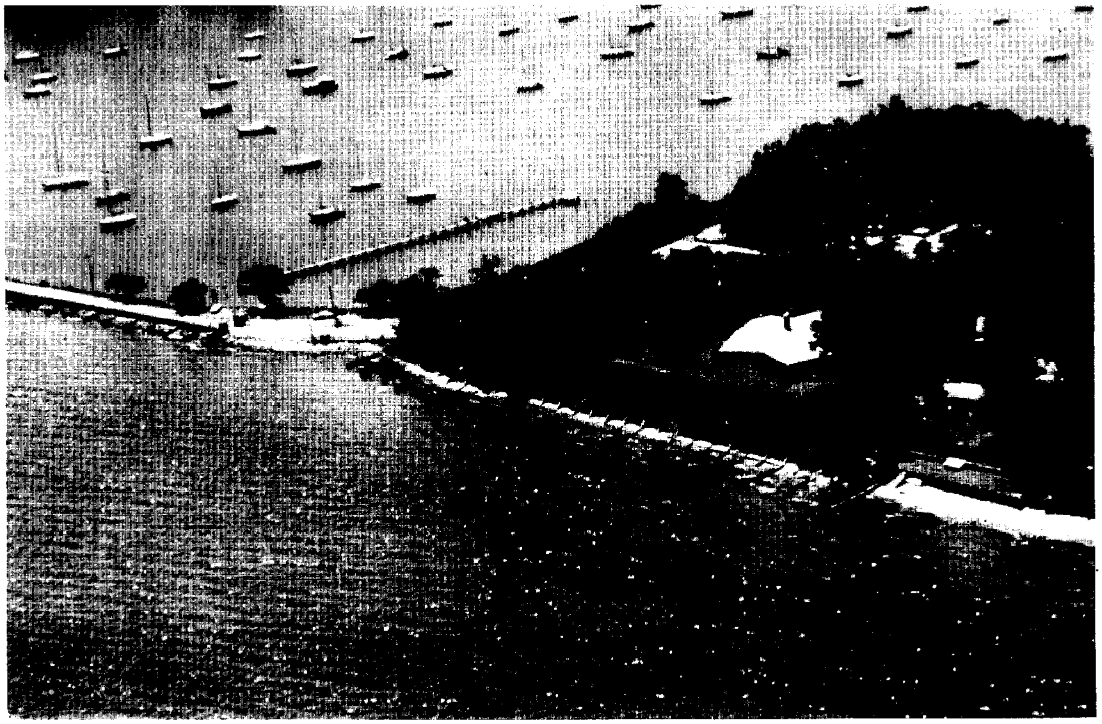
CASE 25 TIMBER BULKHEAD, GROINS, AND BUTTRESSED CONCRETE SEAWALL NEAR GIBSON ISLAND

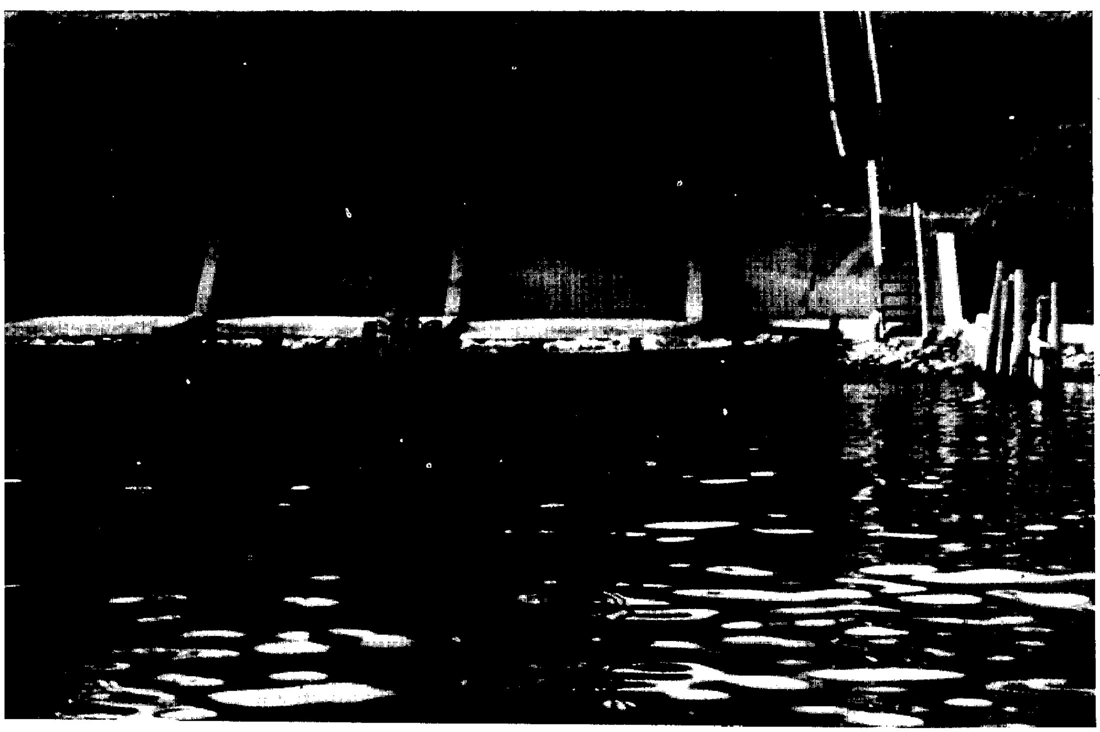
These structures protect a site composed of vegetated bluff. The date and cost of the structures are not known. The historical rate of erosion at the site was less than 1 ft./yr. from 1845-1970.

A variety of structures are present at the site. These include an L-shaped concrete seawall, buttressed with triangular concrete sections. A splashover apron 8 feet wide has also been installed on the landward side. This concrete seawall is separated from a timber bulkhead alongshore at one end. Approximately 9 feet of shoreline separating the two structures is filled with rip-rap. The timber bulkhead is fronted with both rip-rap and timber groins approximately 20 feet long.

These structures are in fairly good condition. Some maintenance work could improve the wall in some locations. The groins have not trapped substantial amounts of sand to form a beach.



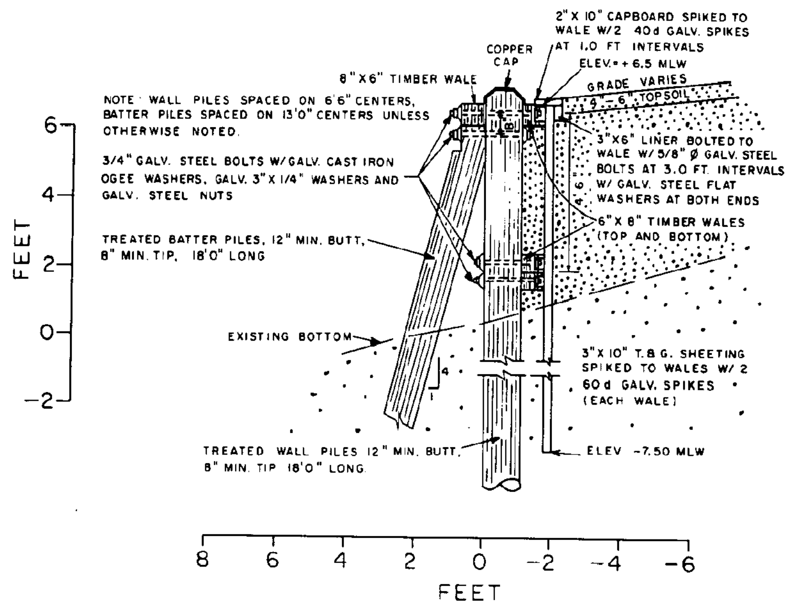


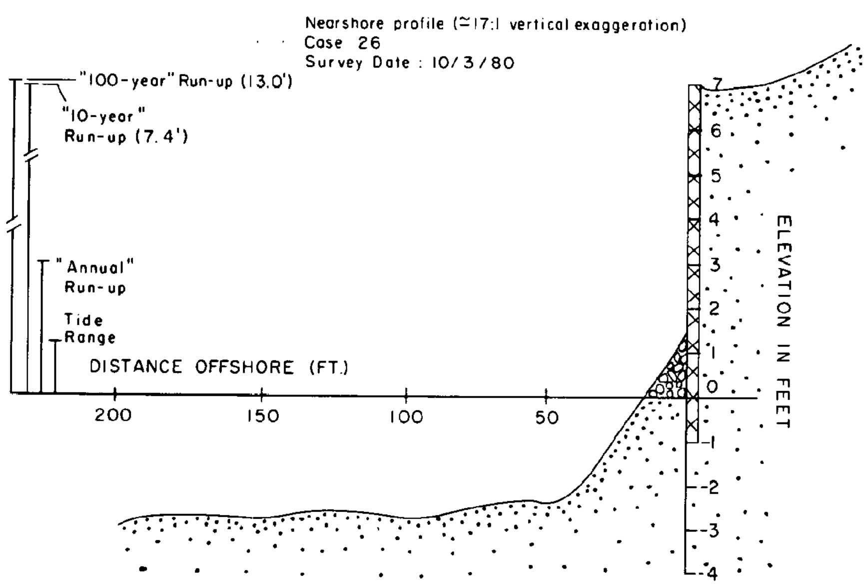


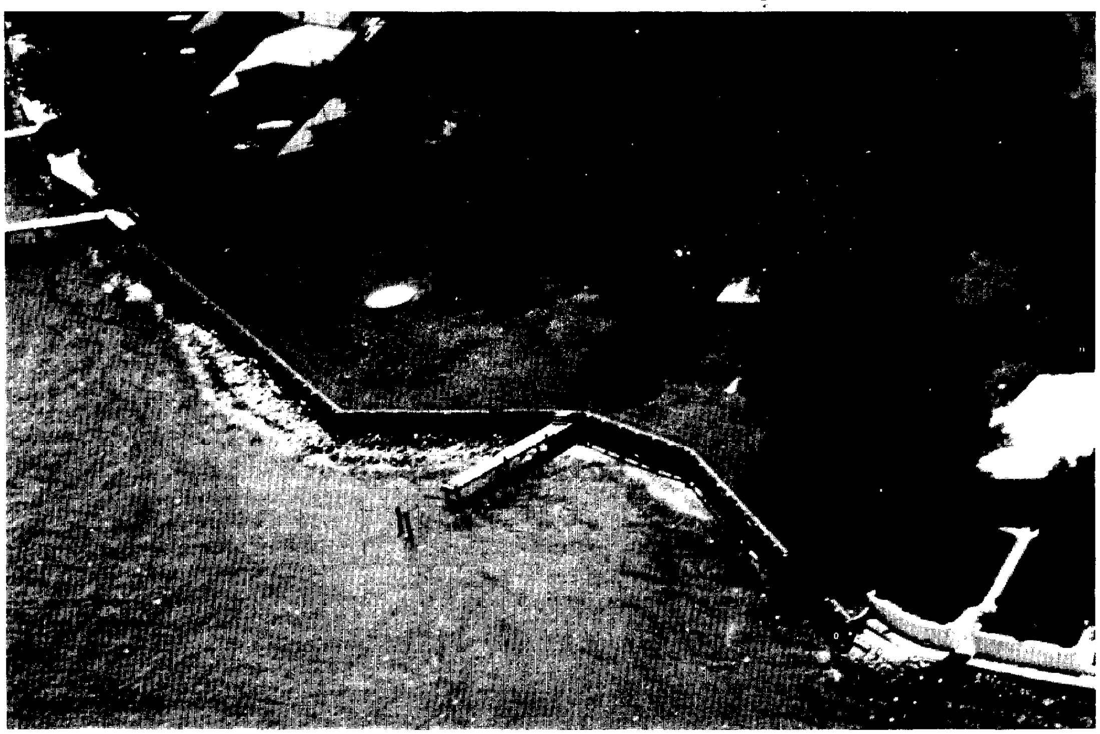
CASE 25 TIMBER BULKHEAD, GROINS, AND BUTTRESSED CONCRETE SEAWALL NEAR GIBSON ISLAND

CASE 26 A TIMBER BULKHEAD AND STONE REVETMENT ON MIDDLE RIVER

Structure was completed in 1978 at a cost of $115.60/ft. The historical rate of erosion at the site was about 2.5 ft./yr. from 1847-1936. Timber bulkhead consists of 3 in. x 10 in. tongue-in-groove sheetpile, and walers 6 in. x 8 in. The planform of the wall is S-shaped. A 10 ft.-wide rip rap toe has also been installed at the base of the structure on the seaward side. This structure is in generally excellent condition. The wall is constructed to extend very high up the bank face and limits splashover from the worst wave conditions. The rip rap at the toe of the structure provides added protection against wave scour and erosion of finer-grained sands and silts of the natural bottom.







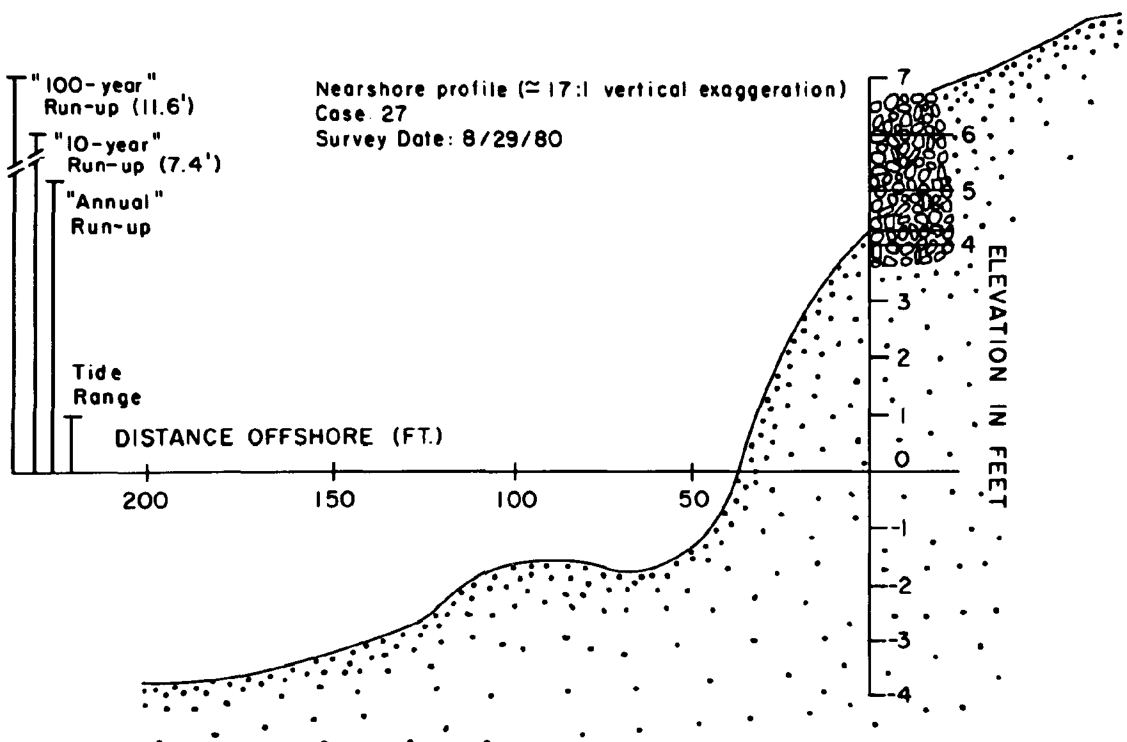
CASE 26 A TIMBER BULKHEAD AND STONE REVETMENT ON MIDDLE RIVER

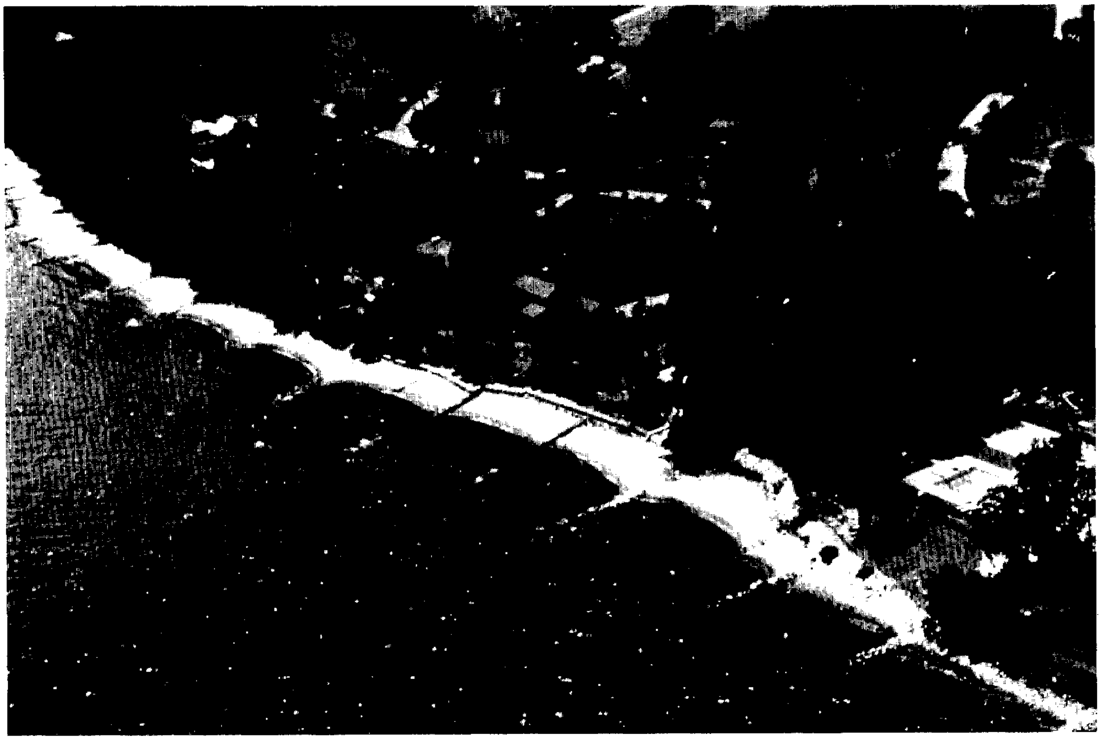
CASE 27 GABIONS ON GIBSON ISLAND WITH GROINS

These structures protect a site composed of a low bank that is eroding in some spots, and covered with trees, shrubs, and grass in others. The historical rate of erosion at the site was about 2-3 ft./yr. from 1845-1970.

The groins were completed in 1970 at an unknown cost. Gabion structures have recently been installed to form a revetment at the base of the bank. Two groins, also composed of gabions, are in place offshore.

This structure is new and had not experienced severe conditions at the time of the site visit. The gabions extend far up the bank face to protect against waves and tidal flooding from all but the worst conditions for this shoreline reach.

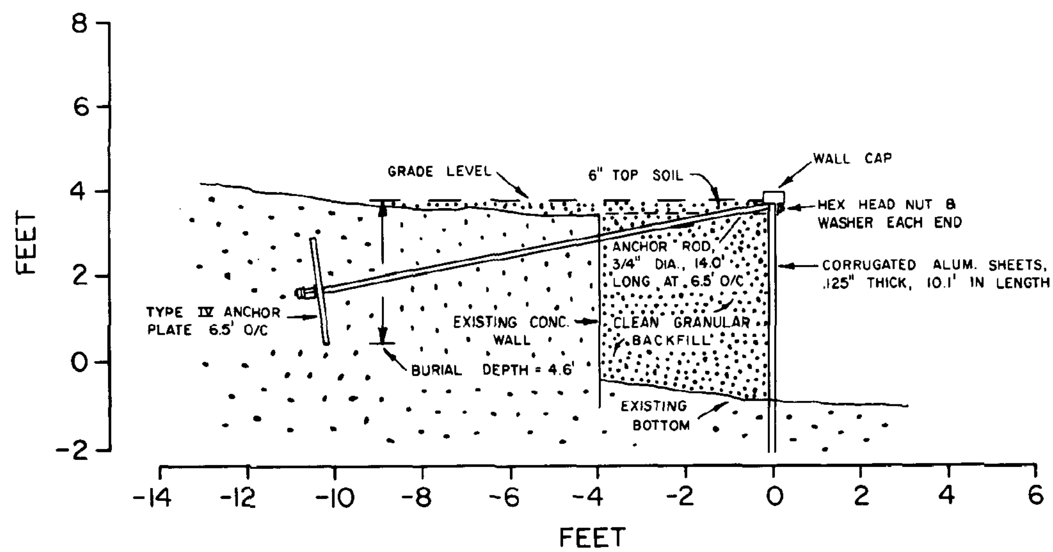


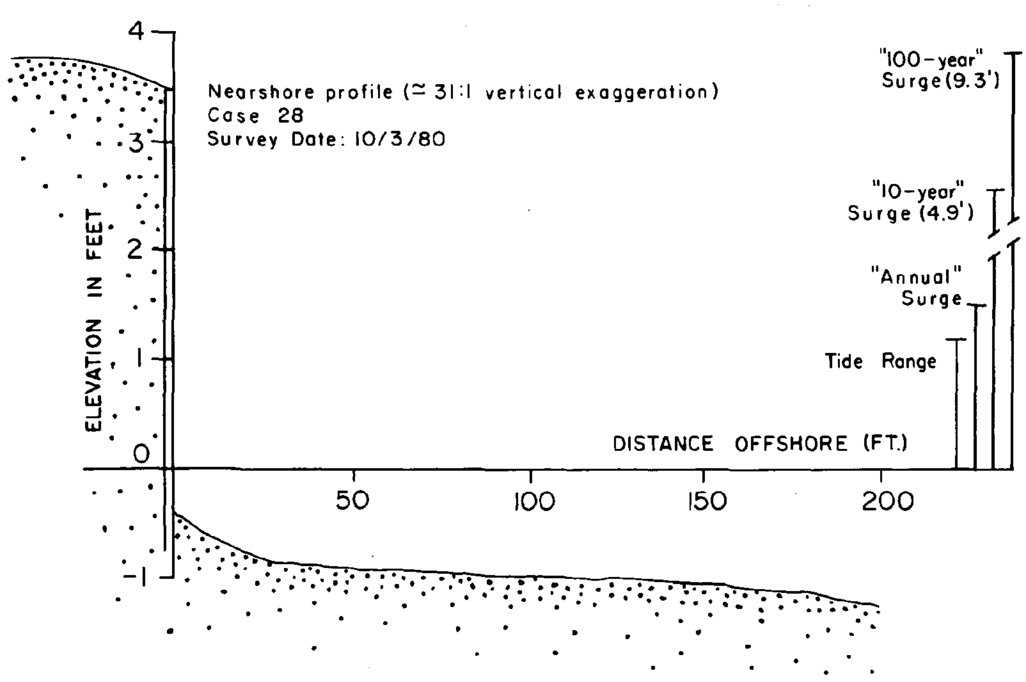


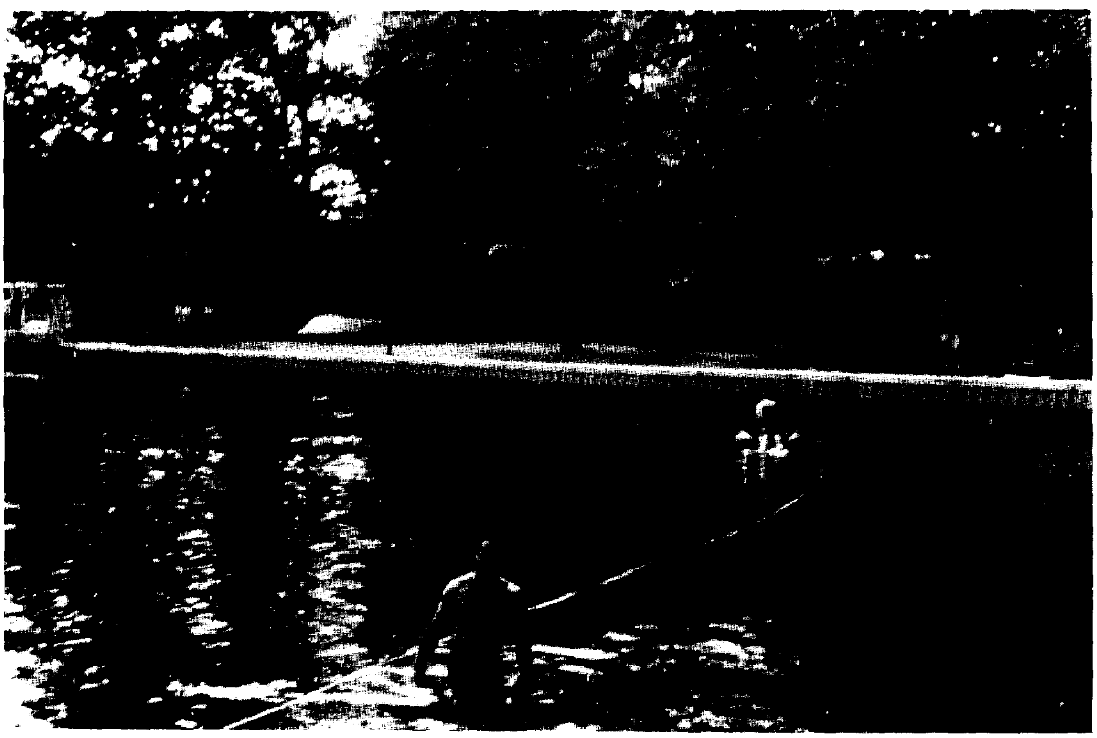
CASE 27 GABIONS ON GIBSON ISLAND WITH GROINS

CASE 28 ALUMINUM BULKHEAD ON BACK RIVER

Structure was completed in 1974 at a cost of $70.00/ft. The historical rate of erosion at the site was about 2.5 ft./yr. from 1846-1944. Structure consists of corrugated aluminum sheetpile 0.125 in. thick and 10.1 ft. long. A deadman anchoring system is connected to the wall by tie rods 3/4 in. in diameter and 14 ft. long. This structure is in generally good condition. There is evidence of splashover at the site, but there are no problems with flanking erosion alongshore since the wall connects with other vertical structures on both ends.







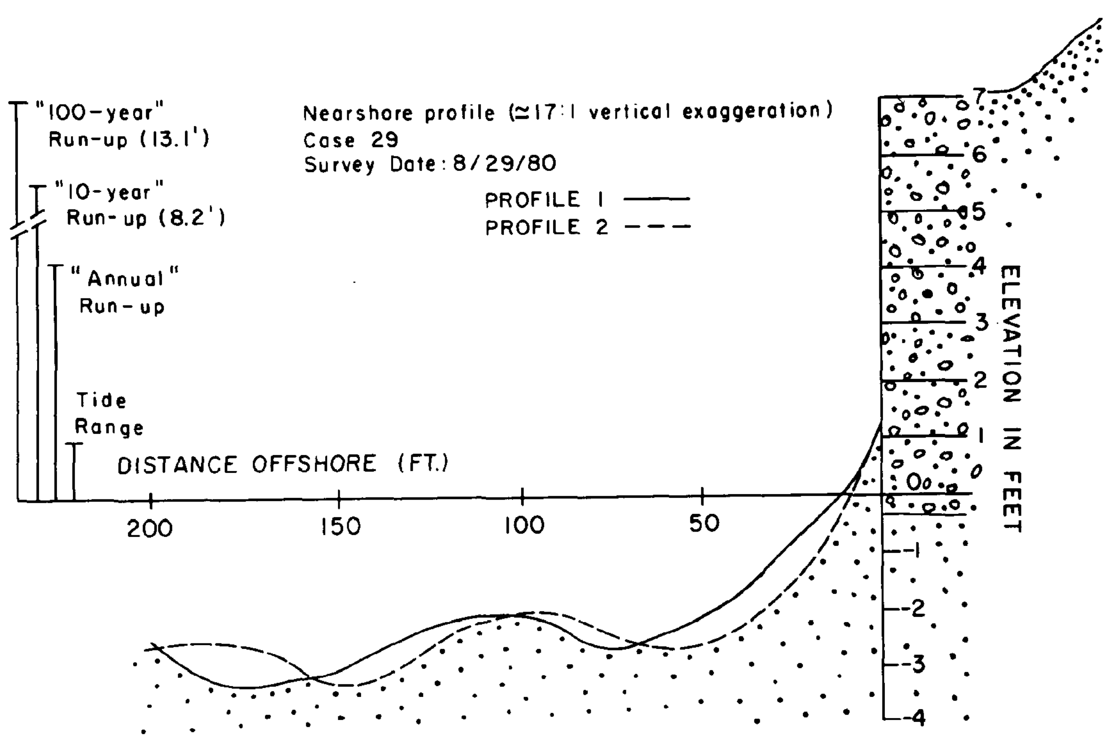
CASE 28 ALUMINUM BULKHEAD ON BACK RIVER

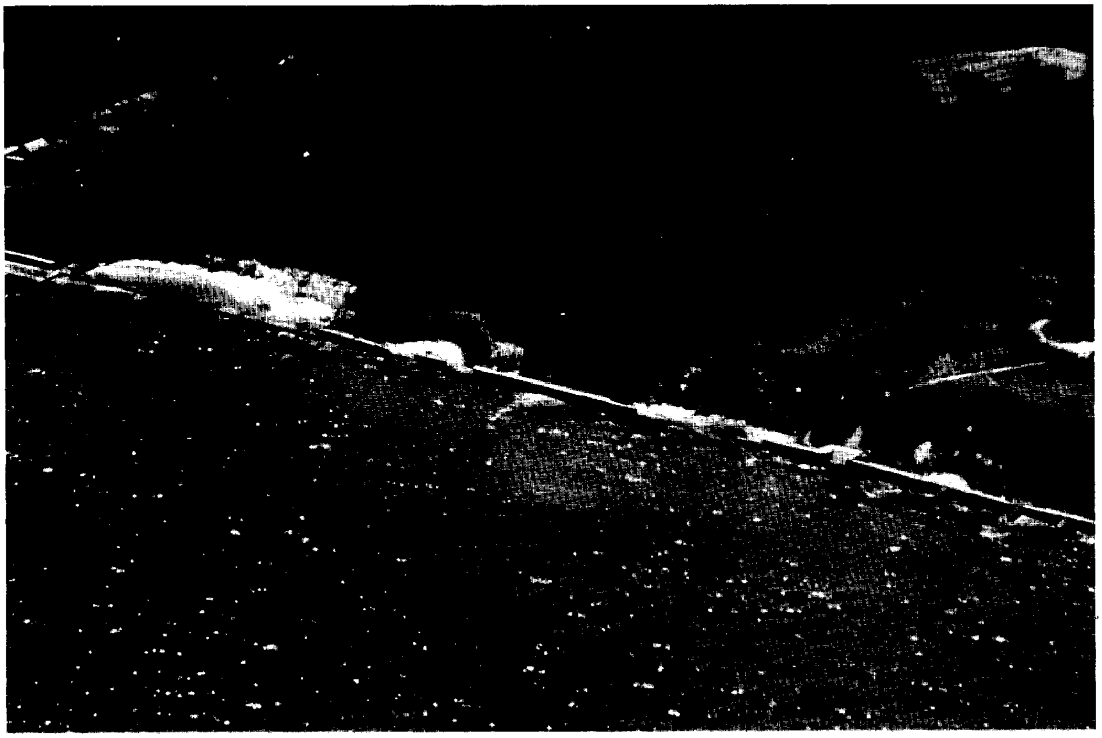
CASE 29 A CONCRETE BULKHEAD NEAR GIBSON ISLAND

This structure protects a site composed of eroding bluff. The historical rate of erosion at the site was less than 1 ft./yr. from 1845-1970.

The structure was completed in 1929-1931 at an unknown cost. Structure consists of concrete seawall approximately 7 feet high. At least two different pours of concrete were used to build the wall, and the different sections are joined by rebars. Stone groins have been implaced at a few locations in front of the seawall.

This structure is in poor condition. The rebars have failed in several places, and this failure has contributed to general wall failure. The groins have accumulated some sand to form a beach offshore, but the amounts of sand in each groin pocket are very small.





CASE 29 A CONCRETE BULKHEAD NEAR GIBSON ISLAND

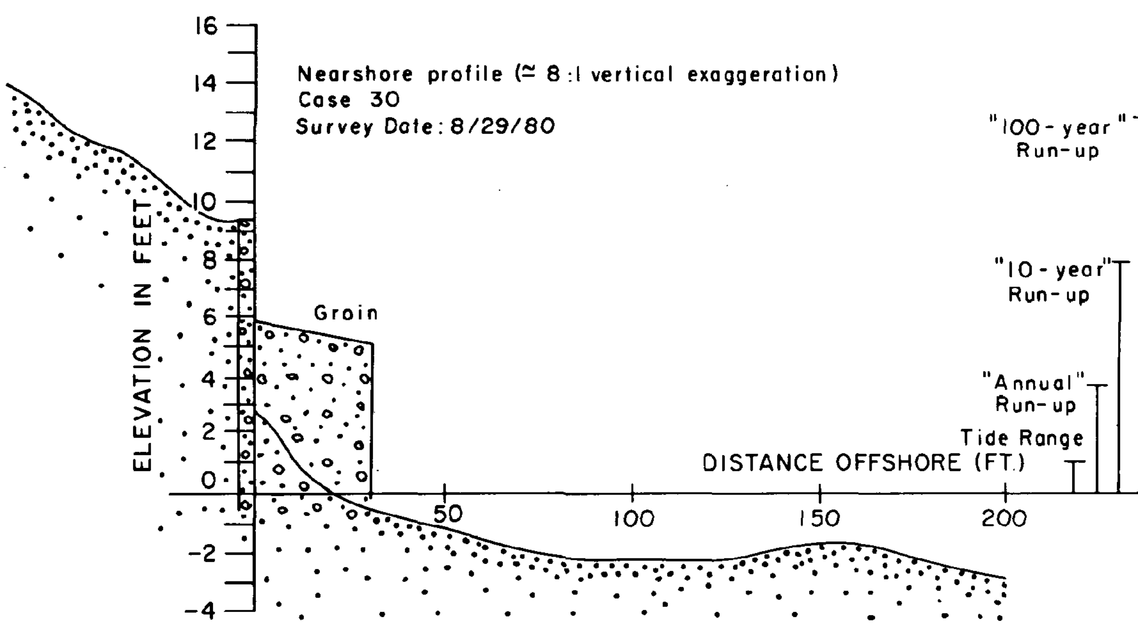
CASE 30 CONCRETE BULKHEAD AND GROINS NEAR HAWKINS POINT

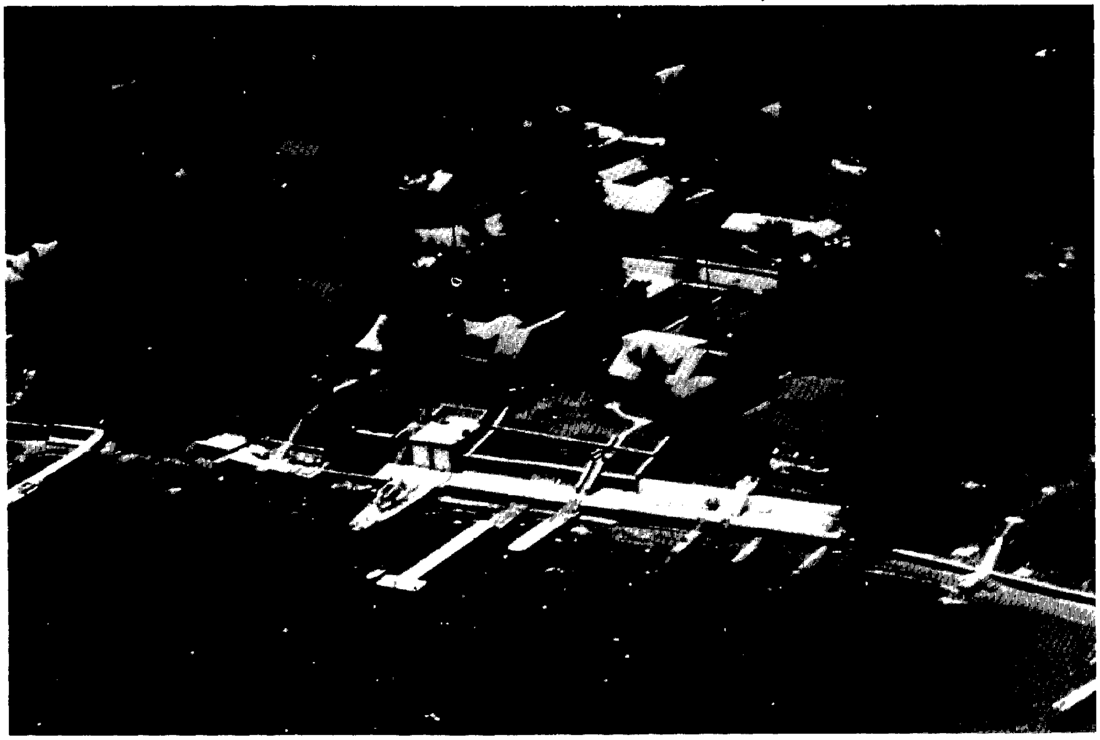
These structures protect a site composed of a high terraced bank. The historical rate of erosion at the site was less than 2 ft./yr. from 1845-1970. The date and cost of the structures is not known.

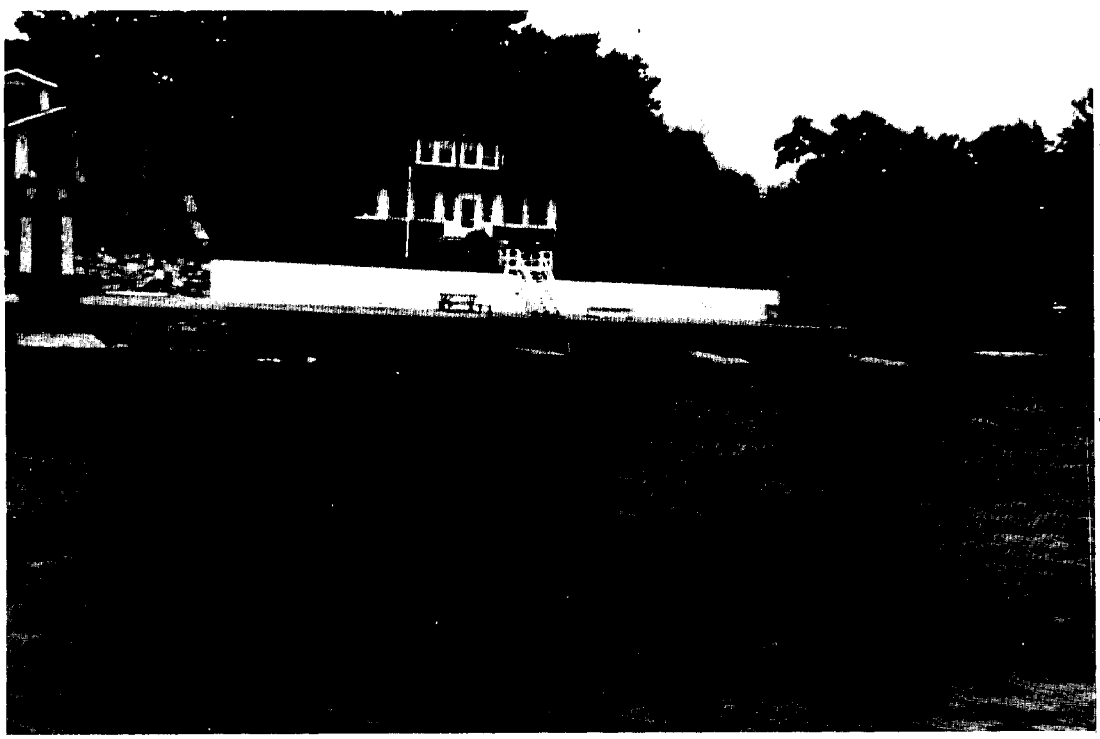
Structures consist of reinforced concrete seawall with a concrete splash-over apron. Three tiers of concrete walls are stacked up the slope of the shoreline bank. The last tier is about 20 feet above the water level.

A masonry dock and groin system has also been installed at the base of the wall. The groins are spaced about 30 feet apart.

These structures are in generally good to fair condition. The structure evidently suffered some damage (which has been repaired) from a 1956 hurricane. Presently, there is freeze/thaw damage requiring the owner to periodically repoint the masonry walls. The groins have not trapped any amounts of sand from the natural littoral drift to form a beach.







CASE 30 CONCRETE BULKHEAD AND GROINS NEAR HAWKINS POINT

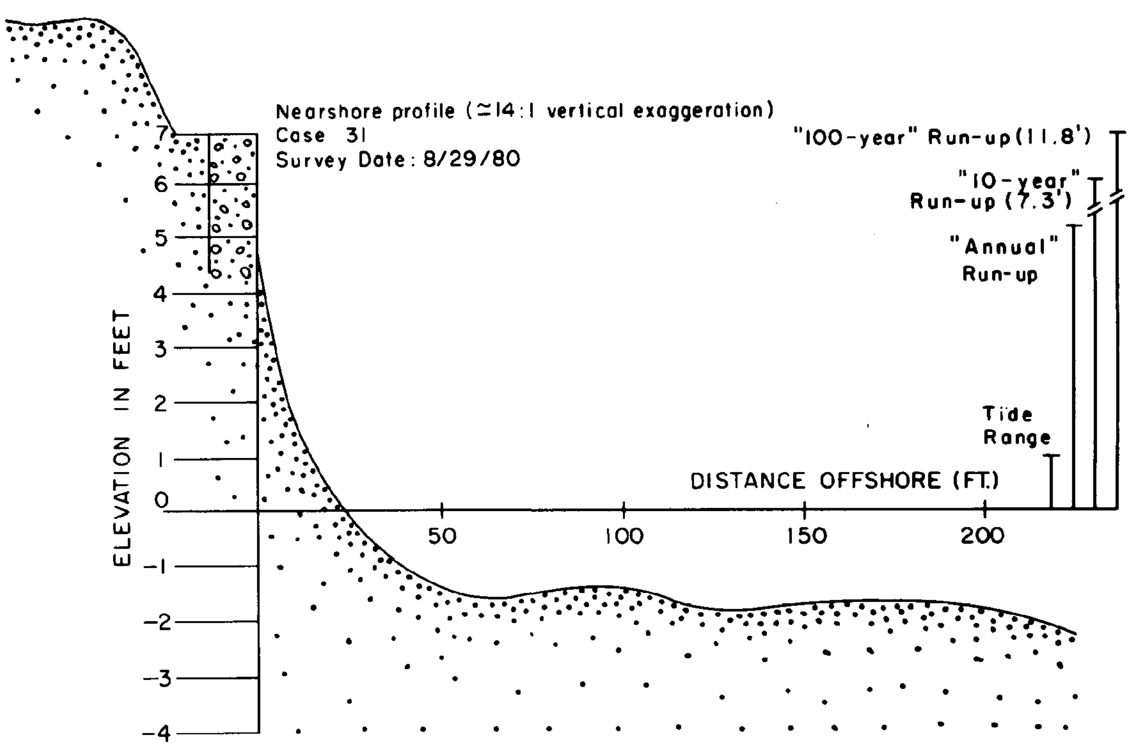
CASE 31 A WELL-RING BULKHEAD ON BROAD NECK

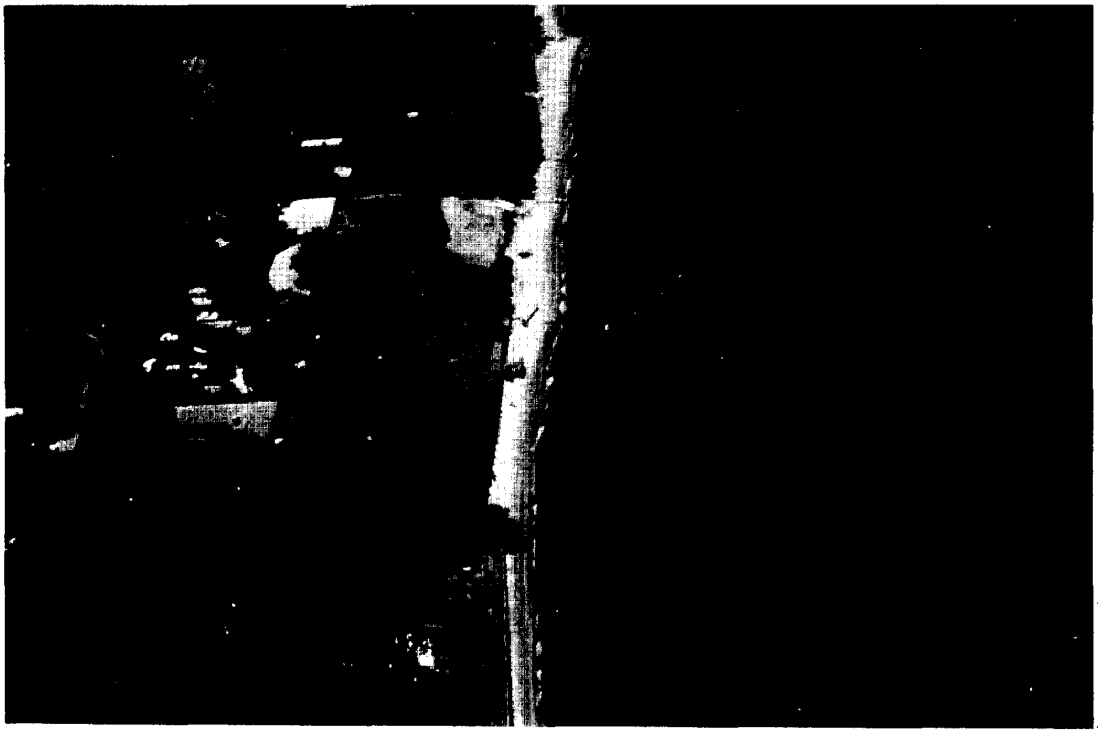
This structure protects a site composed of high sloping bank. The historical rate of erosion at the site was about 2 ft./yr. from 1845-1970. The structure was completed in 1968 at an unknown cost.

The concrete well-rings have an inside diameter of 3 feet. They are placed 3-high, and are filled with stone. Only the top well-ring projects out of the sand.

Two well-ring groins are also installed, with the well-ring bulkhead holding the fastland. The groins were filled with a substantial amount of beach sand at the time of the site visit.

The well-ring bulkhead is in generally good condition. The owners report no problems, and they appear to be quite pleased with the performance of the structure. There is no evidence of splashover or of washing out of the fastland behind the well-ring structure.





CASE 31 A WELL-RING BULKHEAD ON BROAD NECK

F. Cases along the Upper Eastern Shore of Chesapeake Bay

This area of the northern Chesapeake Bay shoreline (Figure 2.10) contains portions of Cecil County and Kent County. The sections below present a brief physical description of the shoreline and coastal processes, followed by a discussion of the case studies which were selected from this area.

SHORELINE DESCRIPTIONS

Eastern Cecil County The Cecil County shoreline along the Elk River is composed of rolling hills up to 80 feet in height. At most spots, the land slopes gently down to the water, but some hillsides end in exposed vertical walls of eroding sediments. The beaches in this area are of varying width, and contain some gravel. Enough sand is present in some areas of the shoreline system to form wide sandy berms on the summer beach profiles. Marshes are found in protected coves, and in the headwaters of the Elk River.

Most shorefront areas on the western side of the Elk River contain farmlands or woodlands. Houses are located in among the trees on hillsides next to the shore, on low banks protected by erosion-control structures, or in open grassy areas behind a wide vegetated buffer zone.

On the eastern side of the Elk River, the terrain is generally flatter, and shorefront areas contain banks ranging from 5 to 15 feet in height. Areas near the entrance to the C&D Canal contain concentrated residential shorefront development and networks of erosion-control structures which stretch nearly continuously along the shore. Other areas contain farmlands or woodlands perched on exposed eroding banks. A beach is present along most of this shoreline, with small pocket marshes found in protected coves and at the heads of the inflowing rivers. The beach profile is widest on isolated points of land. South of the entrance to the C&D Canal there are few structures and few areas of shorefront development. Only woodlands and farmlands are found on the Bohemia and Sassafras Rivers, except at the towns shown on the map.

Further south along the Elk River, there is shorefront residential development at different spots from Crystal Beach to Grove Point. Some houses and vacation cottages are located on rolling hills which slope gently down to the water, and others are found in among stands of trees, next to the edges of exposed bluffs.

Kent County The Kent County shoreline along the Chesapeake Bay consists of bluffs and high banks between 10 and 50 feet in height which overlook the water. From Betterton to Worton Point, the bluffs are exposed and eroding in many areas. A sand and gravel beach of varying width is found at the water's edge. Landward of the beach, there are broad accumulations of eroded slumped material (talus slopes) which make the bluff faces a bit less steep than at other points in the northern Chesapeake Bay. A few isolated points of land have low grassy terraces extending out into the water from the higher ground.

Next Pages: Figure 2.10. Shoreline along Eastern Cecil County and Kent County.

Figure 2.11. Some representative shoreline profiles collected in the summer of 1981 along the upper reaches of the Eastern Shore.

Figure 2.10

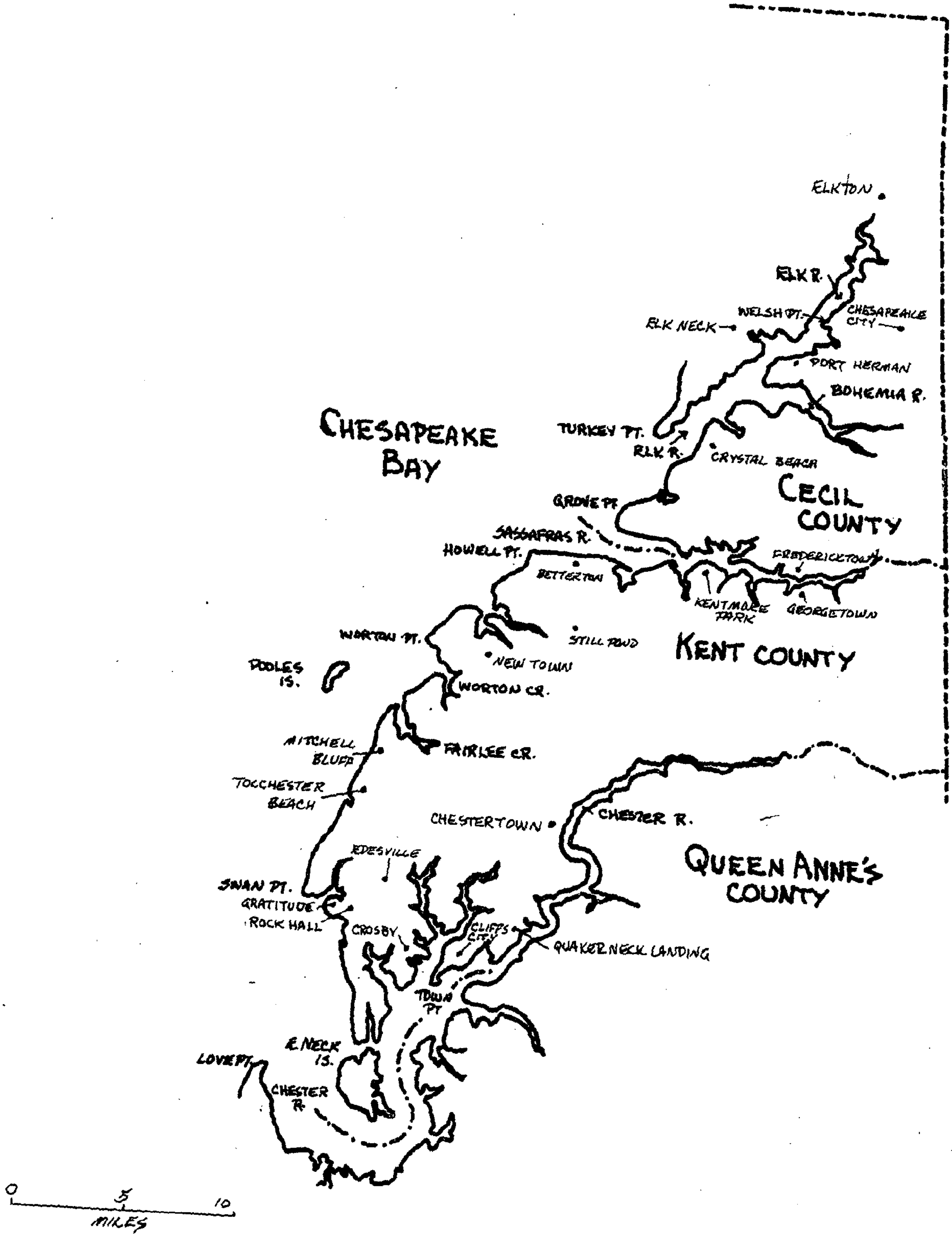
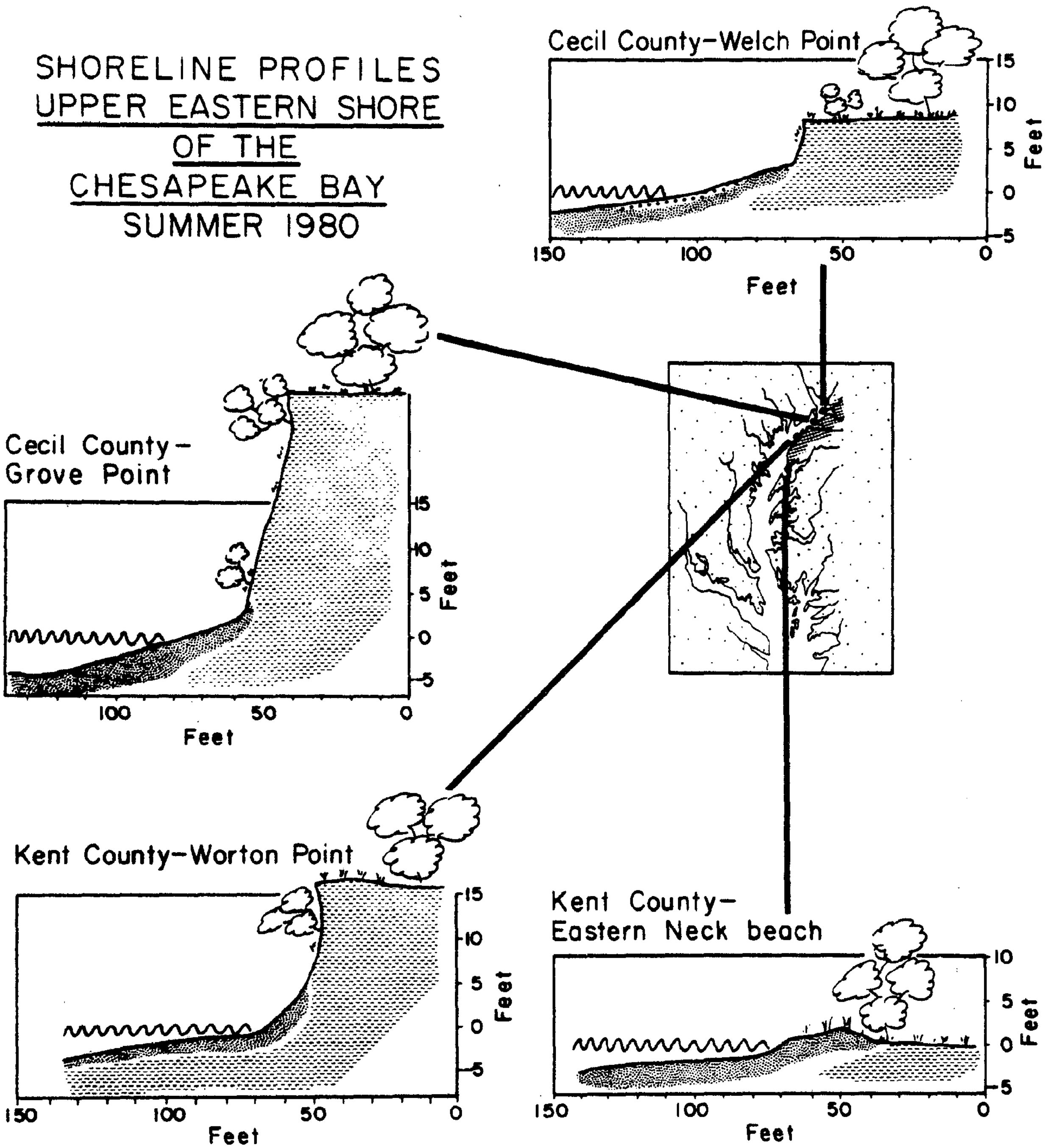

ELKTON
ELK R.
WELSH PT.
CHESAPEAKE CITY
ELK NECK
PORT HERMAN
BOHEMIA R.
CHESAPEAKE BAY
TURKEY PT.
ELK R.
CRYSTAL BEACH
CECIL COUNTY
GROVE PT
SASSAFRAS R.
HOWELL PT.
BETTERTON
FREDERICKTOWN
KENTMORE PARK
GEORGETOWN
WORTON PT.
STILL POND
KENT COUNTY
NEW TOWN
POOLES IS.
WORTON CR.
MITCHELL BLUFF
FAIRLEE CR.
TOLCHESTER BEACH
CHESTERTOWN
CHESTER R.
EDESVILLE
QUEEN ANNE'S COUNTY
SWAN PT.
GRATITUDE
ROCK HALL
CROSBY
CLIFFS CITY
QUAKER NECK LANDING
TOWN PT
E. NECK IS.
LOVE PT.
CHESTER R.
0
5
10
MILES

Figure 2.11


SHORELINE PROFILES
UPPER EASTERN SHORE
OF THE
CHESAPEAKE BAY
SUMMER 1980
Cecil County-Welch Point
15
10
5
0
-5
Feet
150
100
50
0
Feet
Cecil County-
Grove Point
15
10
5
0
-5
Feet
100
50
0
Feet
Kent County-Worton Point
15
10
5
0
-5
Feet
150
100
50
0
Feet
Kent County-
Eastern Neck beach
10
5
0
5
Feet
150
100
50
0
Feet

At Worton Point, the shoreline banks are very steep and end abruptly at the water's edge. The sediments exposed along the shoreline in this area contain considerable clay and the nearshore zone contains a shallow wave-cut shelf of hard slippery clay covered in some spots by a thin layer of sand. Beaches in this area are found only inside protected coves.

From Worton Creek south to Swan Point, the shoreline consists of high banks fronted in most areas by a beach containing both sand and gravel. Most shorefront areas south of Tolchester contain farmlands and woodlands, but from Tolchester north to Betterton, homes are found along the edges of the bluffs and high banks in many spots.

At Rock Hall, the shoreline contains banks of varying height, and there is a wide shallow nearshore area that is sometimes exposed at times of low tide. The shoreline fronting the Chesapeake Bay below Rock Hall is composed of low rolling hills fronted by beaches which stretch nearly continuously along the shore. In some spots the beach is backed by low dunes stabilized by grasses and shrubs.

Coastal Processes The historical erosion rates for this portion of the northern Chesapeake Bay shoreline range from 2-8 ft./year. There are a few sites where the shoreline is stable or accreting. The shoreline sediments which are eroded are sandy deposits of the Monmouth, Matawan, and Raritan Formations (Appendix A). The sediments in littoral drift are moving in both directions along the shoreline, and the potential rates of littoral drift are variable depending on the different shoreline orientations.

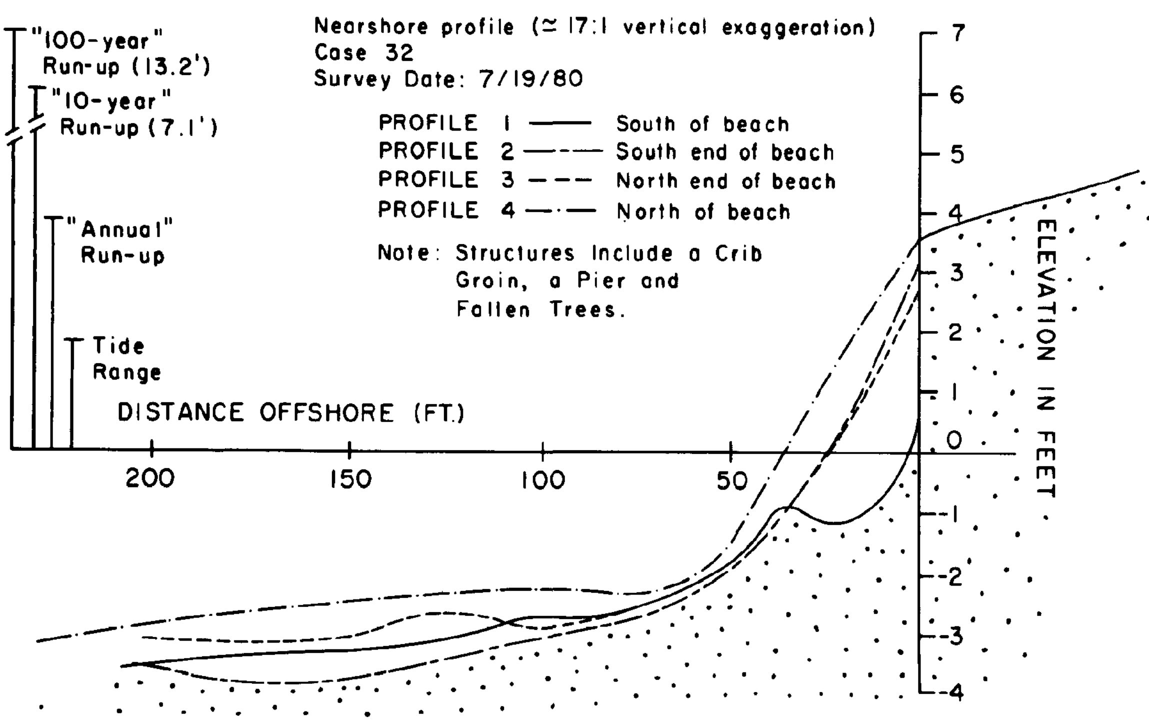
Waves in this area approach from the northwest and southwest with the longest fetches. Only protected coves are sheltered from wave energy by

CASE 32 GROINS AT BETTERTON BEACH

These structures protect a site composed of sandy beach fronting a low sandy berm and vegetated bank on the Betterton waterfront. The historical rate of erosion at the site was less than 3 ft./yr. from 1845-1947. The cost and date of the structures are not known.

The pocket beach is held between a groin to the west alongshore and a crib jetty/pier, 250 feet long, to the east. Groins are constructed of both stone and timber.

The structures are in generally fair condition. The beach is largely fill material, and erosion is occurring to the west alongshore from the beach. But the existing pocket beach between the two structures appears to be held reasonably well, despite the deteriorated nature of the structures.



irregularities in the shoreline, and the shallow offshore bottom runs in a very narrow zone between the beach and the navigation channel along the Eastern shore. As a result, wave energies are medium to high for this entire shoreline reach.

The mean tide range is above 1 foot in most areas. The storm surge from "annual storms" is about 1 foot, and the surges from "100 year" storms can be greater than 8 feet in many coastal segments. The wave and storm conditions are discussed in more detail along with the other coastal processes in Chapter V.

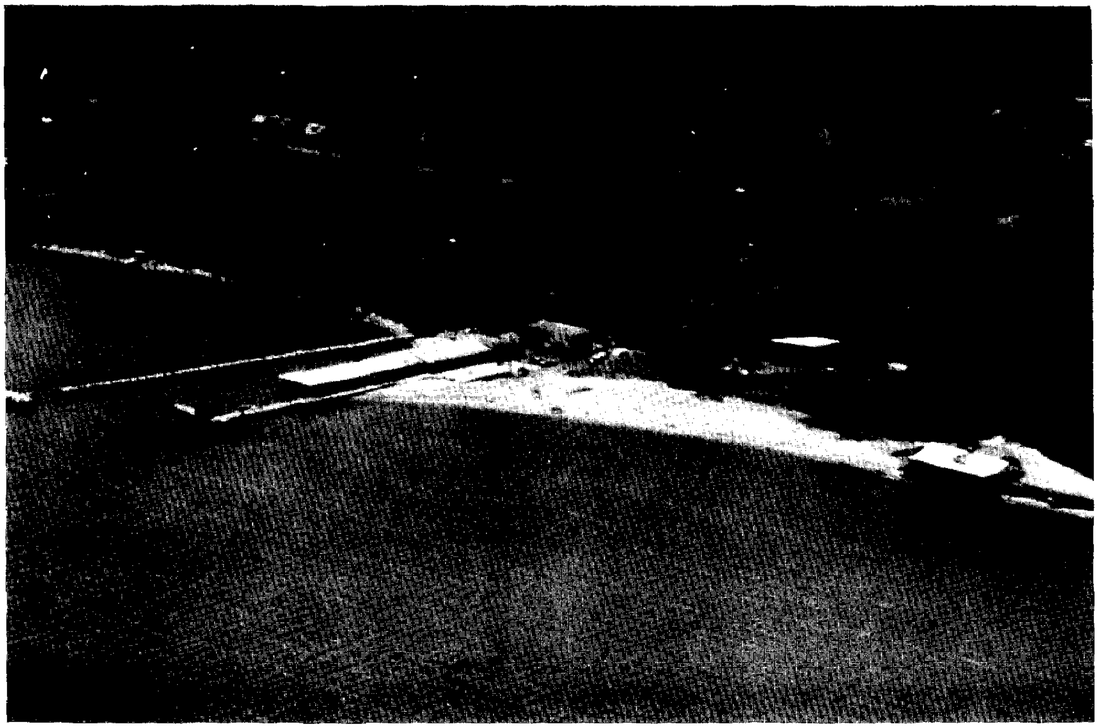
Cases The structure case studies selected in this area include:

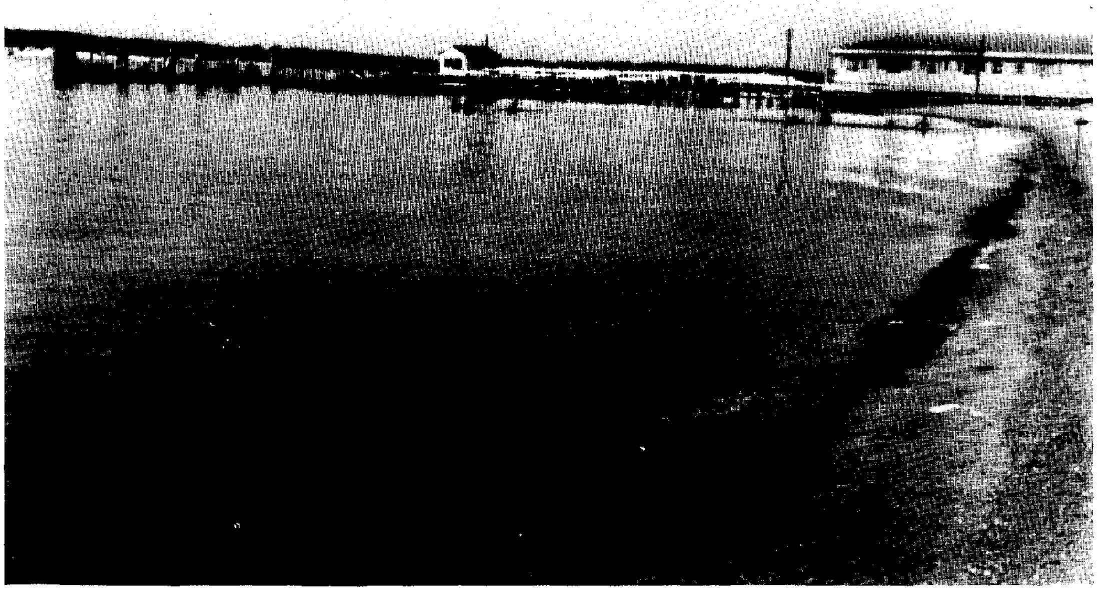
Case No.	Structure
32	Groins at Betterton Beach.
33	Stone revetment at Mitchell Bluffs (372.5 feet long).

The following pages present a brief description of each structure and nearshore bottom profiles collected at the sites. Both cases are in generally fair condition. There is a beach at both sites, and the revetment at Mitchell Bluffs should protect against wave splashover from all but the worst wave conditions. A redesign of the structure at Betterton Beach is discussed below.

Redesign of Betterton Beach

The continued existence of a pocket beach at Betterton requires the nearly complete retention of sand within the beach "compartment" formed by the groin near the beach club building to the west and the crib pier to the east. The groin should be more sand-tight and extended up higher to prevent westerly sand transport. The beach can then be maintained in good condition. To the west there is active erosion and means to curtail it should be explored.





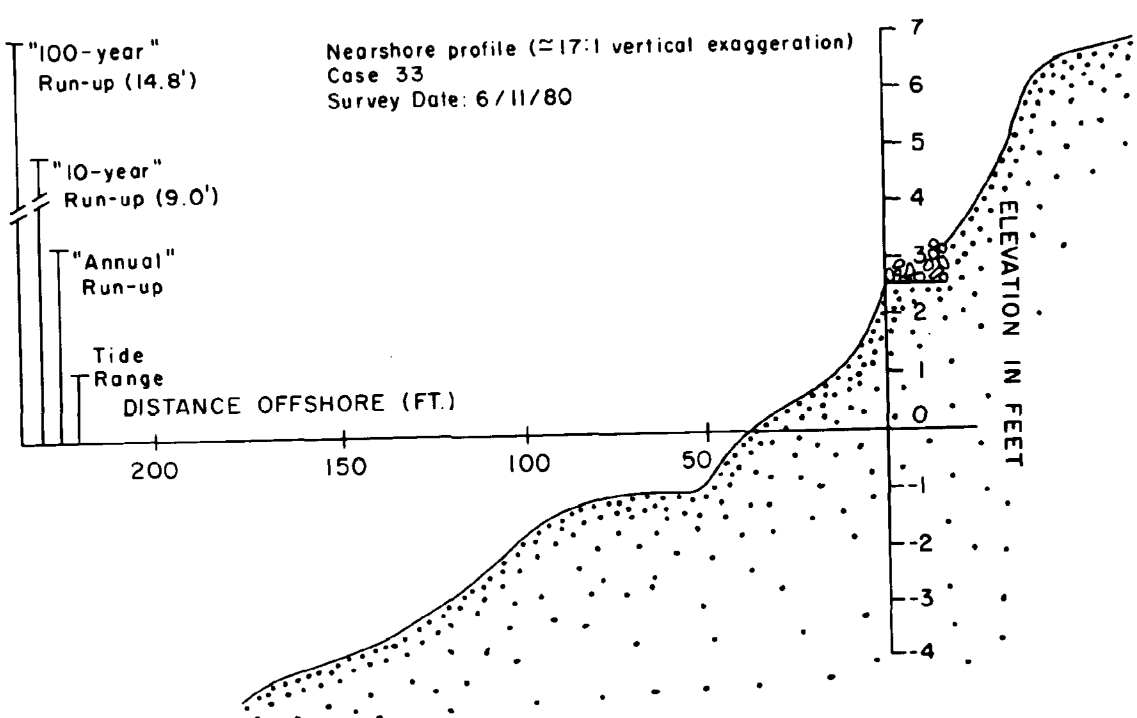
CASE 32 GROINS AT BETTERTON BEACH

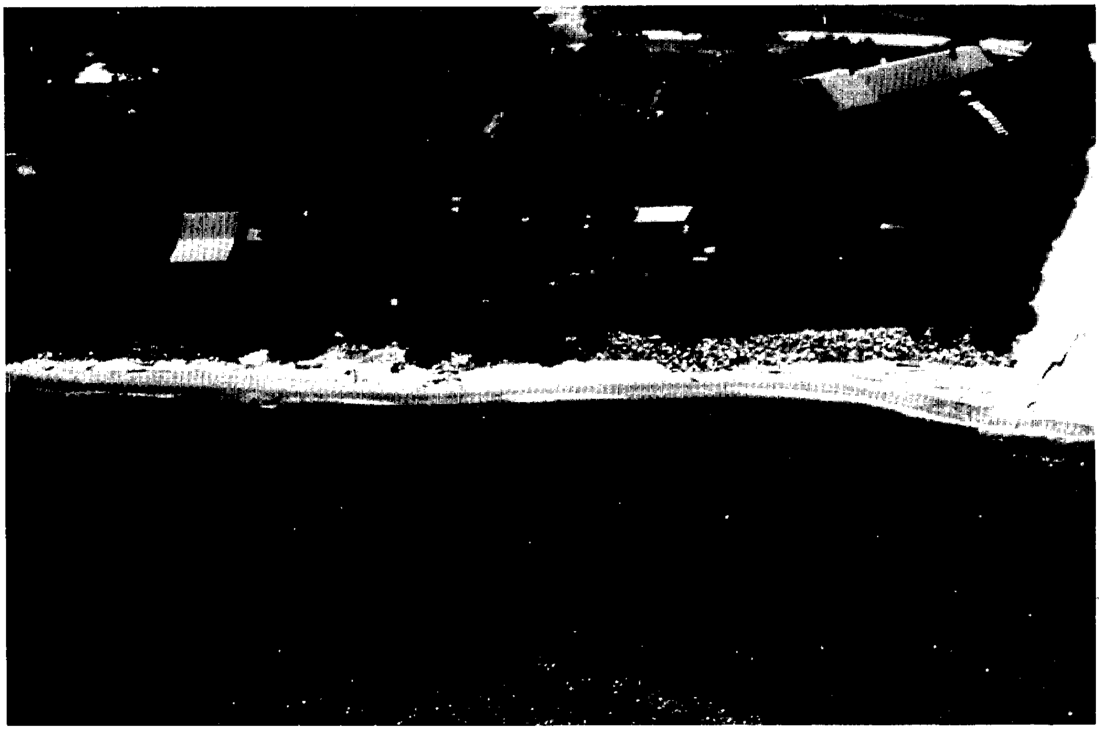
CASE 33 A STONE REVETMENT AT MITCHELL BLUFFS

This structure protects a site composed of high eroding bank, alongshore from high bluffs. The historical rate of erosion at the site was about 1 ft./yr. from 1845-1953.

The structure was completed in 1975 at a cost of $60.48 per foot. Revetment is in two sections, separated by a gap. Armor layer is composed of 350-1200 lbs. stone, 2 feet-thick, overlying a 6 inch-thick bedding layer of quarry stone. Filter cloth was used below the bedding layer. The revetment has a maximum elevation of 7 feet above MLW.

This structure is in generally fair to good condition. A sandy beach exists along the shoreline reach, and the offshore bottom is covered with stones. There is significant shoreline retreat and active erosion presently alongshore to the north of this site. This adjacent eroding shoreline was once protected by a timber bulkhead which has since failed.





CASE 33 A STONE REVETMENT AT MITCHELL BLUFFS

G. Cases along Kent Island and Talbot County Shorelines

This area of the northern Chesapeake Bay (Figures 2.12 and 2.13) contains portions of the shoreline in Queen Anne's and Talbot Counties. The sections below present a brief physical description of the shorelines and coastal processes, followed by a discussion of the case studies which were selected from this area.

Kent Island The Queen Anne's County shoreline on Kent Island contains farmland, a few heavily-wooded areas, and many clusters of shorefront homes protected by erosion-control structures. North of the Bay Bridge, the Kent Island shoreline is composed of low rolling hills which usually end at the water's edge in exposed banks from 3 to 15 feet in height. An exception is at Love Point, where the high ground slopes gently down to the water's edge. Shorefront areas on this reach contain mostly farmland, and the few houses that are present are separated from the water by a wide vegetated buffer strip.

Below the Bay Bridge, many of the shoreline banks support residential development, and shorefront areas are often landscaped, covered with lawn grass and shrubbery, and protected by erosion-control structures. Clusters of homes are interspersed with tidal creeks surrounded by marsh. Farther south, the shoreline contains farmlands and woodlands at the edges of eroding banks, and the beach is nearly uninterrupted by erosion-control structures. The few groin fields that are present were filled with sand in the summer of 1980.

The southern most portion of Kent Island is a low wooded terrace extending out from higher ground, and fronted by marsh and small pocket beaches. A few homes are present along the shore, protected by erosion-control structures.

Talbot County From Tilghman Point south to Wades Point, the shoreline consists mostly of exposed eroding banks between 3 and 15 feet high. There is a beach at the base of the banks in most spots, and the trees perched on heavily-wooded Tilghman Point are falling off the banks onto the beach.

Farther south along this shoreline reach, shorefront areas contain farmlands, and low-lying terraces containing grass, shrubs, and marsh, that extend out into the water from higher ground. Concentrated shorefront development and erosion-control structures are present on the shore at Claiborne, and at many different locations south along the shore to Lowes Point.

From Lowes Point south to Tilghman Island, shorefront areas contain mostly woodlands, farmlands, and marsh. Where a marsh is absent, the shoreline banks are usually exposed and eroding, and end abruptly at the water's edge.

Residential and commercial shorefront development is concentrated on the shoreline around Knapps Narrows, and some structures have been installed to halt erosion. Along the Bay shoreline south of Knapps Narrows, shorefront areas contain exposed eroding banks which end at the water's

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Next Pages: Figure 2.12. Shoreline fronting the Chesapeake Bay along Kent Island and Talbot County.

Figure 2.13. Some representative shoreline profiles collected in the summers of 1980 and 1981 from Kent Island and Talbot County.

Figure 2.12

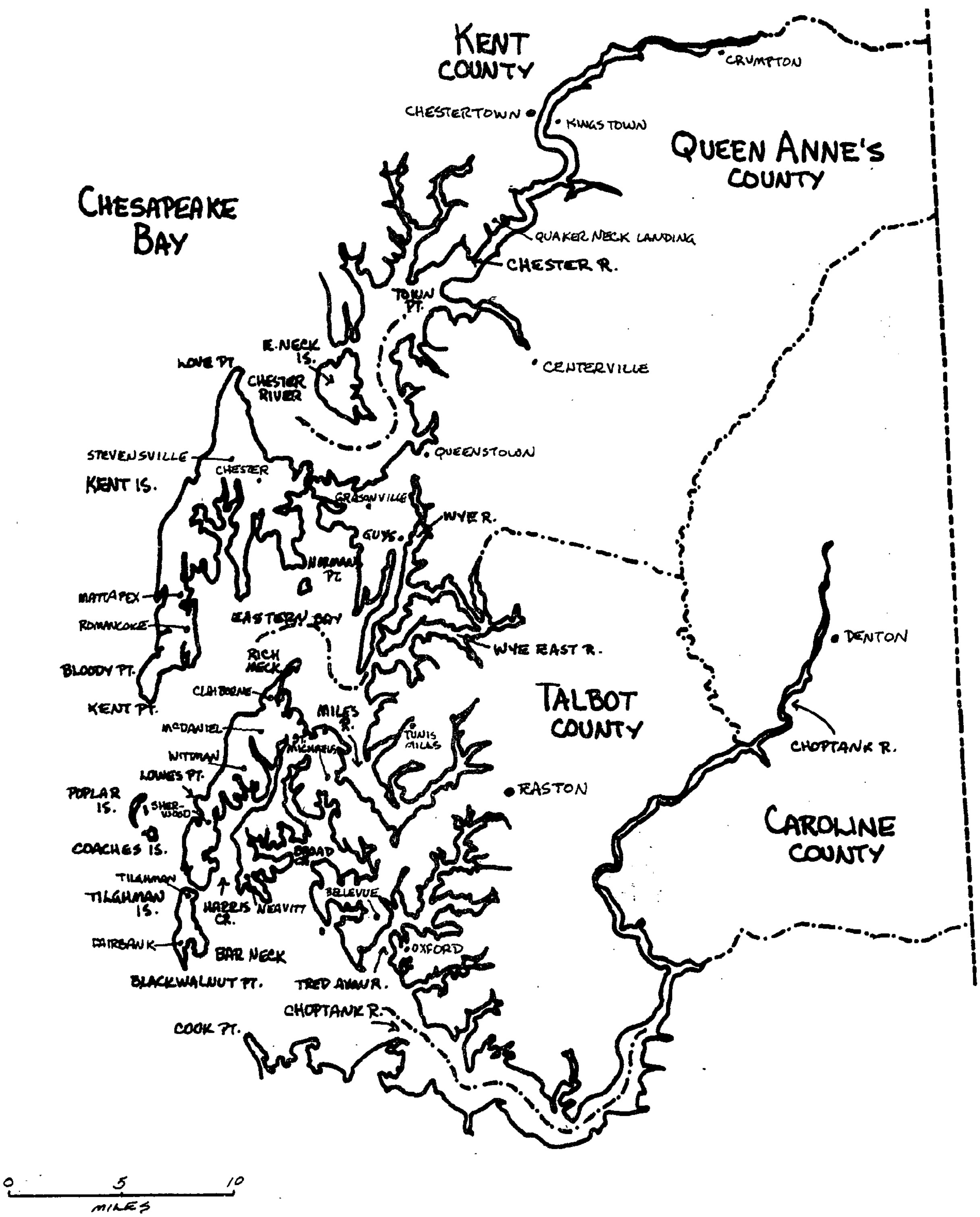
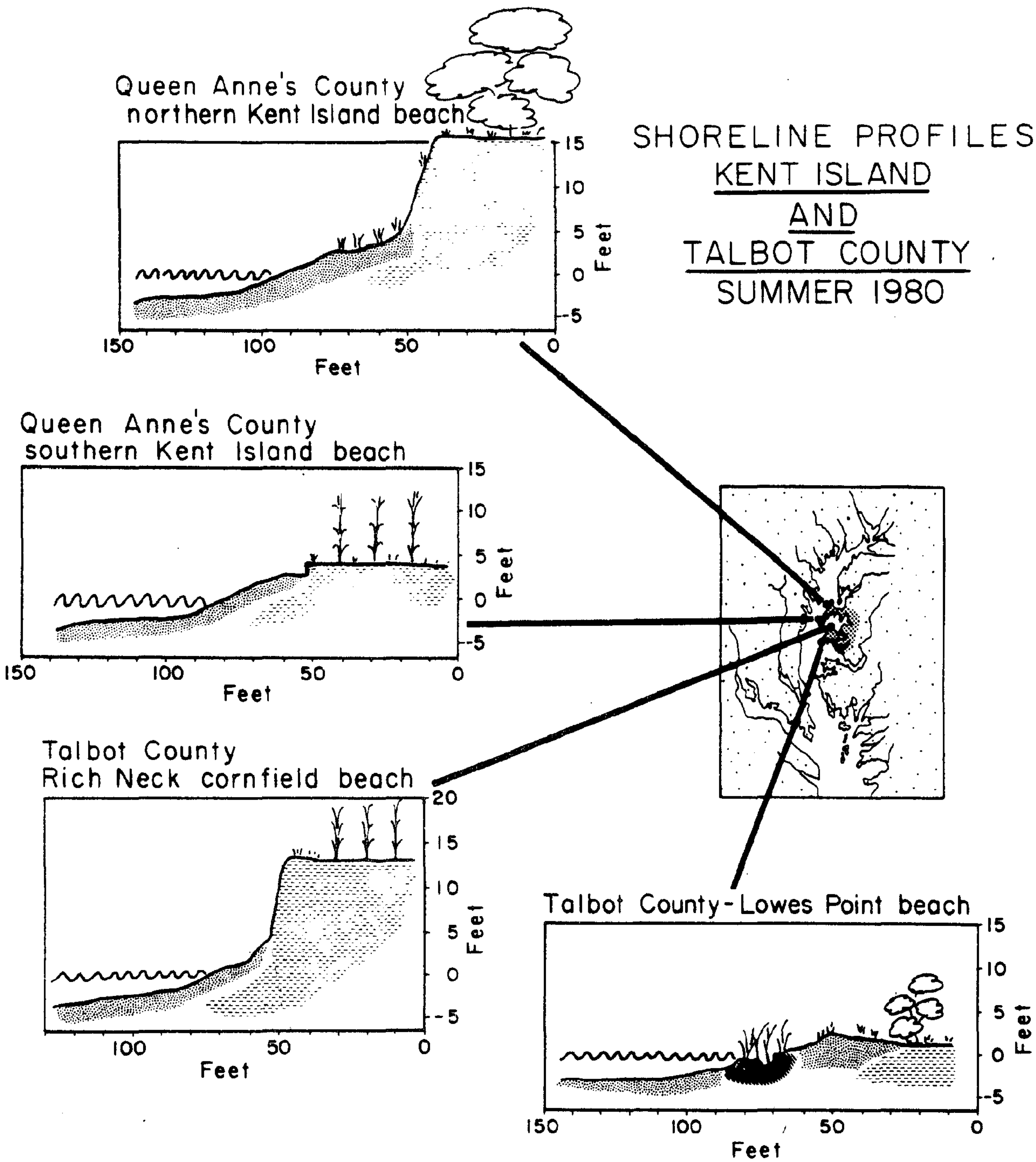

KENT COUNTY
CRUMPTON
CHESTERTOWN
KINGSTOWN
QUEEN ANNE'S COUNTY
CHESAPEAKE BAY
QUAKER NECK LANDING
CHESTER R.
TOWN PT.
E. NECK IS.
LOVE PT.
CHESTER RIVER
CENTERVILLE
STEVENSVILLE
CHESTER
QUEENSTOWN
KENT IS.
GRASONVILLE
WYE R.
GUYS
NORMAN PT.
MATTAPEX
ROMANCOKE
EASTERN BAY
WYE EAST R.
DENTON
RICH NECK
BLOODY PT.
CLAIBORNE
KENT PT.
TALBOT COUNTY
MILES R.
MCDANIEL
TUNIS MILLS
WITTMAN
ST. MICHAELS
CHOPTANK R.
LOWES PT.
EASTON
POPLAR IS.
SHERWOOD
COACHES IS.
CAROLINE COUNTY
BROAD CR.
TILGHMAN
TILGHMAN IS.
HARRIS CR.
NEAVITT
BELLEVUE
FAIRBANK
BAR NECK
OXFORD
BLACKWALNUT PT.
TRED AVON R.
CHOPTANK R.
COOK PT.
0
5
10
MILES

Figure 2.13


SHORELINE PROFILES
KENT ISLAND
AND
TALBOT COUNTY
SUMMER 1980
Queen Anne's County
northern Kent Island beach
15
10
5
0
-5
Feet
150
100
50
0
Feet
Queen Anne's County
southern Kent Island beach
15
10
5
0
-5
Feet
150
100
50
0
Feet
Talbot County
Rich Neck cornfield beach
20
15
10
5
0
-5
Feet
100
50
0
Feet
Talbot County-Lowes Point beach
15
10
5
0
-5
Feet
150
100
50
0
Feet

edge in most spots. In a few places, the higher ground is separated from the water by patches of marsh, and small pocket beaches. Shorefront areas contain farmlands and clusters of houses in open grassy areas.

Along the shore at the southern end of Tilghman Island, and on Poplar and Coaches Island, a beach is narrow or absent, and dead trees are littered in the nearshore zone of breaking waves. This is evidence of rapid shoreline retreat. Most homes on the southern end of Tilghman Island are separated from the water on the Bay side by a narrow buffer strip of roads, and fields or trees, but these same buildings are immediately adjacent to the water on Blackwalnut Cove.

Coastal Processes The historical rates of erosion for this portion of the northern Chesapeake Bay shoreline range from less than 2 ft./yr. to greater than 8 ft./yr. in some areas. The shoreline sediments which are eroded are sandy sediments of several different geologic formations. Sediments in littoral drift are moving in both directions along the shoreline, and the potential rates of littoral drift are variable with different shoreline orientations.

Waves in this area approach with the longest fetches from the northwest and southwest. Shallow offshore areas vary in width along the different coastal segments. Few of these reaches are sheltered from wave energy by irregularities in the shoreline. The wave energies (Table 5.5) are uniformly medium in strength along these reaches.

The mean tide range is about 1.5 feet. Storm surges from "annual storms" are around 3 feet, and the surges from "100 year" storms can be greater than 5-6 feet in many areas.

Cases The structure case studies selected in this area include:

Case No.	Structure
• 34	A stone revetment, timber bulkheads, and stone groins at the south end of Tilghman Island.
• 35	Timber bulkhead on Kent Island (650.5 feet long) with 4 stone groins.
• 36	Stone revetment (360 feet long) with stone groins (160 feet total length) placed seaward of an existing timber bulkhead on Kent Island.
• 37	An experimental site on Tilghman Island.
• 38	Gabions on Kent Island.
• 39	Stone revetment (475 feet long) at Wades Point.
• 40	Concrete pipe on Tilghman Island.

The following pages present a brief description of each structure and nearshore bottom profiles collected at the sites. Most of the case structures assigned in this area were in good condition. A sand beach was noticed in front of some of the structures during the site visits in the summer of 1980. But few of the structures will protect the fastland against wave overtopping during severe conditions.

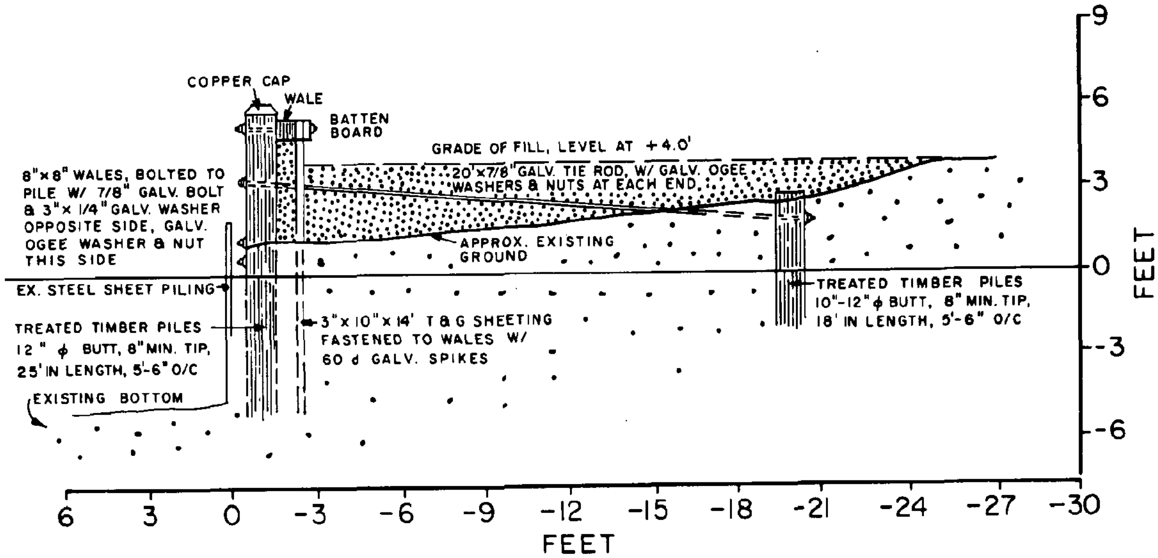
The following paragraphs discuss some design deficiencies at one of the sites.

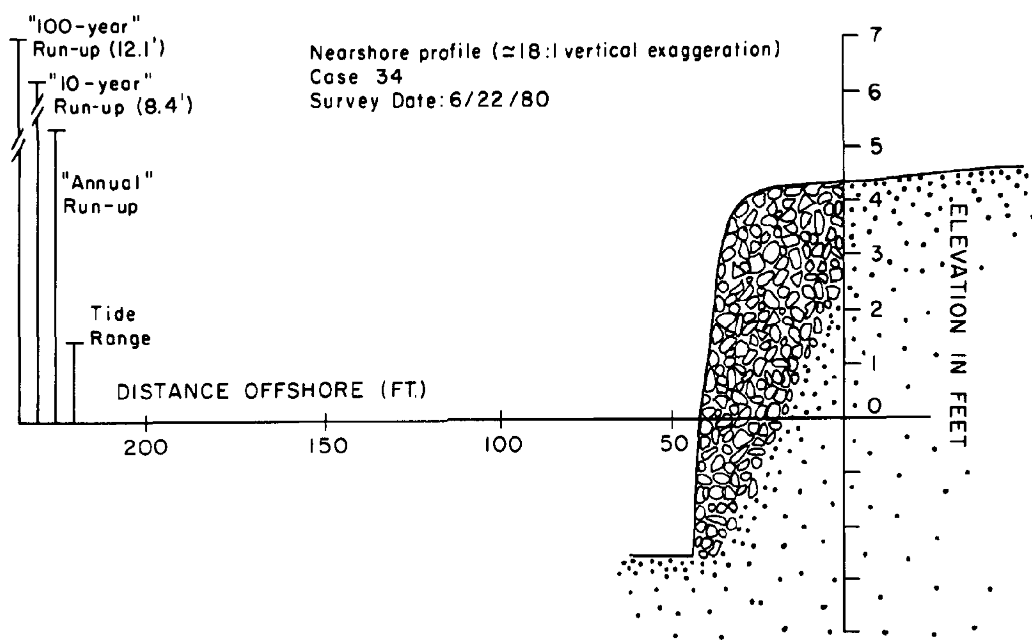
Case 58-73 Timber bulkhead (308 ft.) with stone revetment on Kent Island.

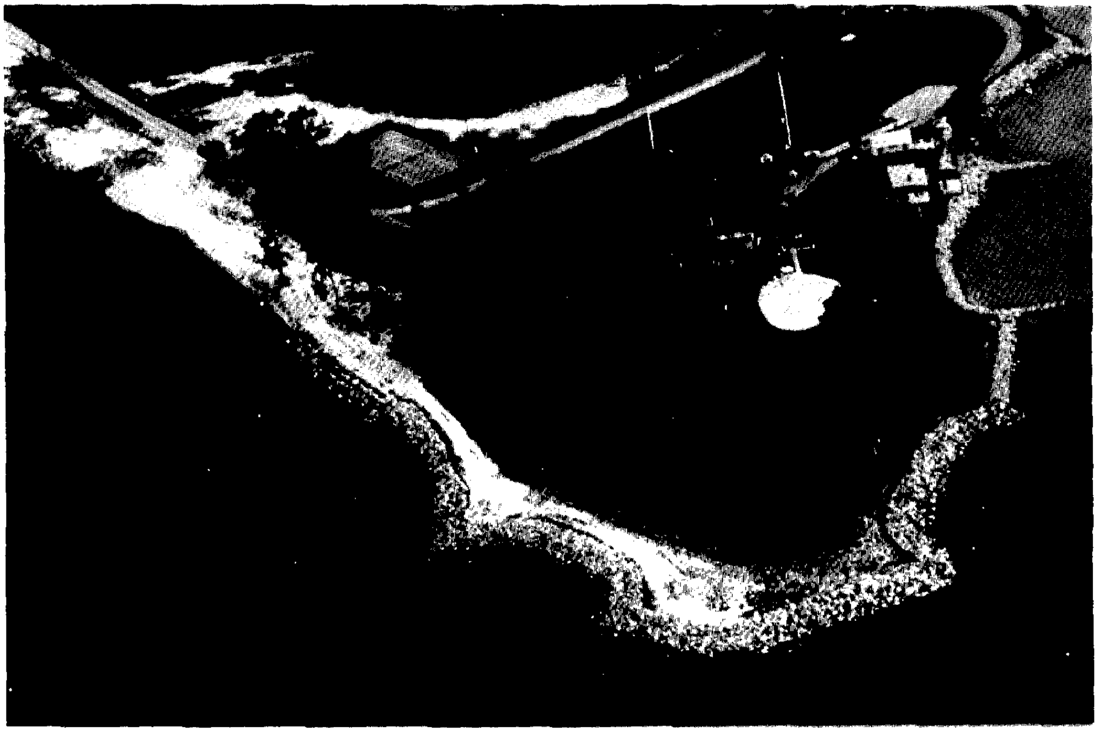
The ponding of water behind the bulkhead and the obvious attendent erosion is evidence of a wall which was too low. Based on an R value of 1 (see Section B in Chapter III), the wall would have to be 1.8 ft. higher to achieve reasonable protection against overwash and splashover. Other aspects of the design appear to be satisfactory.

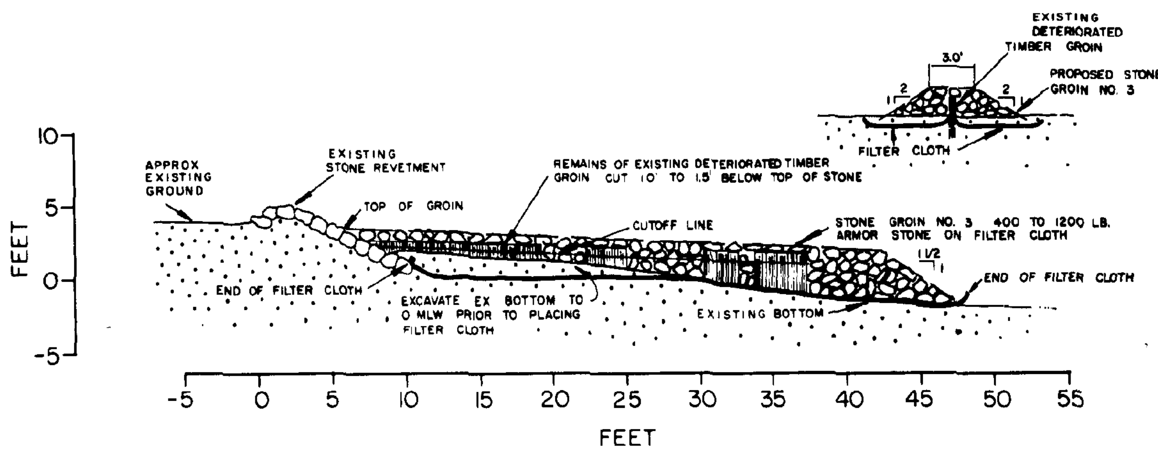
CASE 34 A STONE REVETMENT, TIMBER BULKHEADS, AND STONE GROINS AT THE SOUTH END OF TILGHMAN ISLAND

In 1976, the following work was accomplished: 3 new groins (40 ft., 53 ft., and 63 ft. long) were installed at a cost of $106.17/ft.; timber bulkhead was replaced at a cost of $163.64/ft.; and repairs to stone revetment cost $77.88 /ft. The historical rate of erosion at the site was 8 ft./yr. from 1847-1942. The timber bulkheads replaced sheetpile bulkheads at this site. Stone revetment was installed along with filter cloth. These structures are in generally good condition. The offshore profile is very deep at the base of the seawall, and no beach existed at the time of the site visit in summer of 1980. There is evidence of wave overtopping at locations along the timber bulkheads.





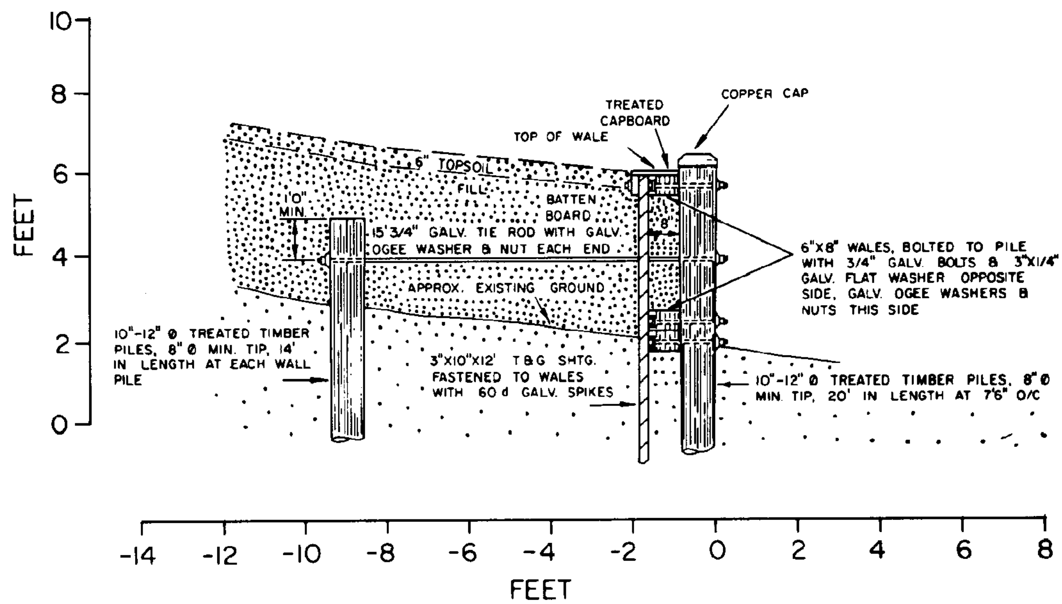


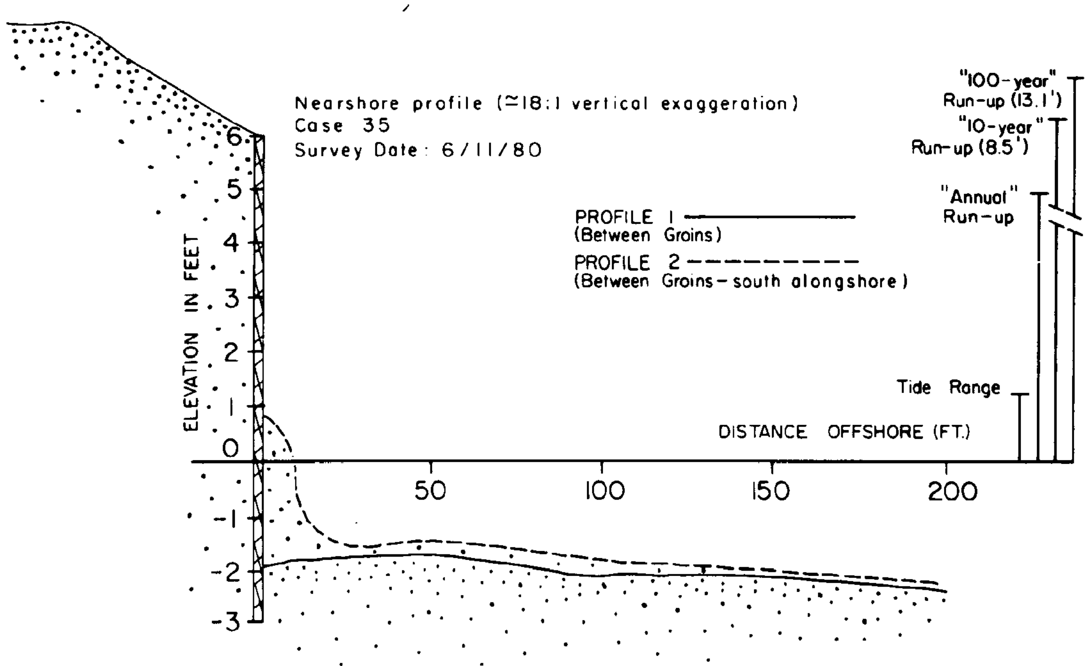


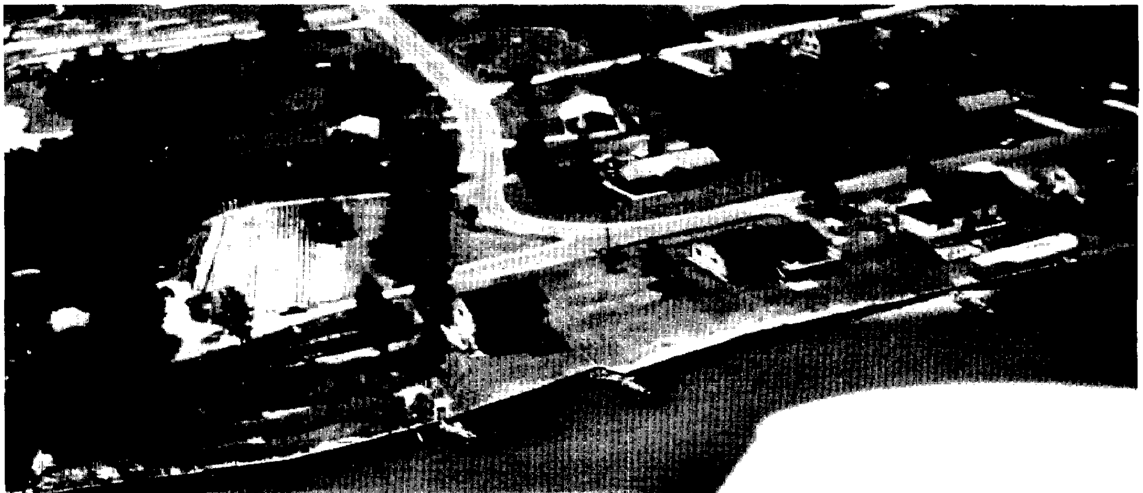
CASE 34 A STONE REVETMENT, TIMBER BULKHEADS, AND STONE GROINS AT THE SOUTH END OF TILGHMAN ISLAND

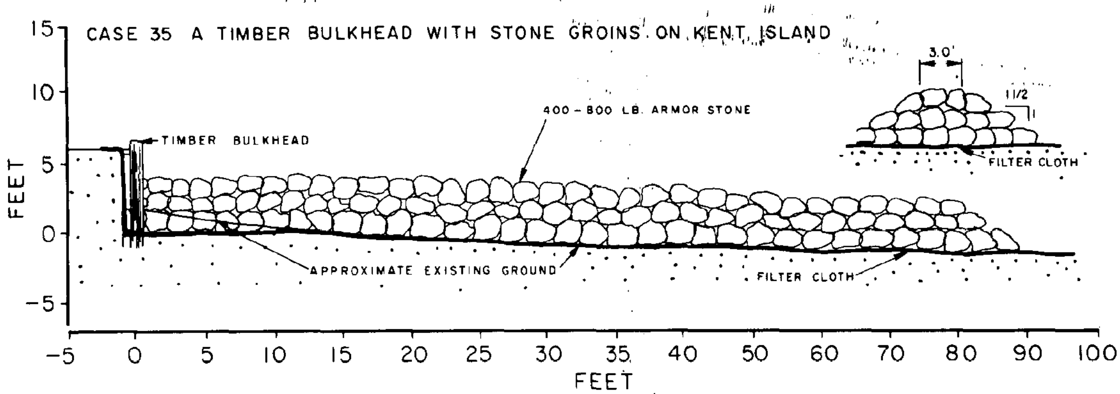
CASE 35 A TIMBER BULKHEAD ON KENT ISLAND WITH STONE GROINS

One stone groin existed prior to the construction of the rest of the structures. The original timber bulkhead and one groin were constructed in 1973 at a cost of $63.29/ft. and $35.80/ft., respectively. In 1976, 493 ft. of timber bulkhead and two stone groins were added at a cost of $79.35/ft. and $54.91/ft., respectively, and the two existing groins were refurbished and extended at a cost of $39.92/ft. Timber bulkheads consist of tongue-in-groove sheeting, and piles are spaced on 7.5 ft. centers. Groins were constructed of 400-800 lbs. stone on a 1.5:1 slope with a 3 ft.-wide crest. These structures are in generally good condition. No beach sand was observed at the site in the summer of 1980. There is strong wave activity in this area, and the wave crests were observed to reflect against the vertical bulkhead. Bulkheads to the north alongshore are being forced landward by waves, and backfill is being lost through the bulkhead wall. There is also evidence of wave overtopping and splashover at this site.









CASE 35 A TIMBER BULKHEAD ON KENT ISLAND WITH STONE GROINS

CASE 36 TIMBER BULKHEAD WITH STONE REVETMENT ON KENT ISLAND

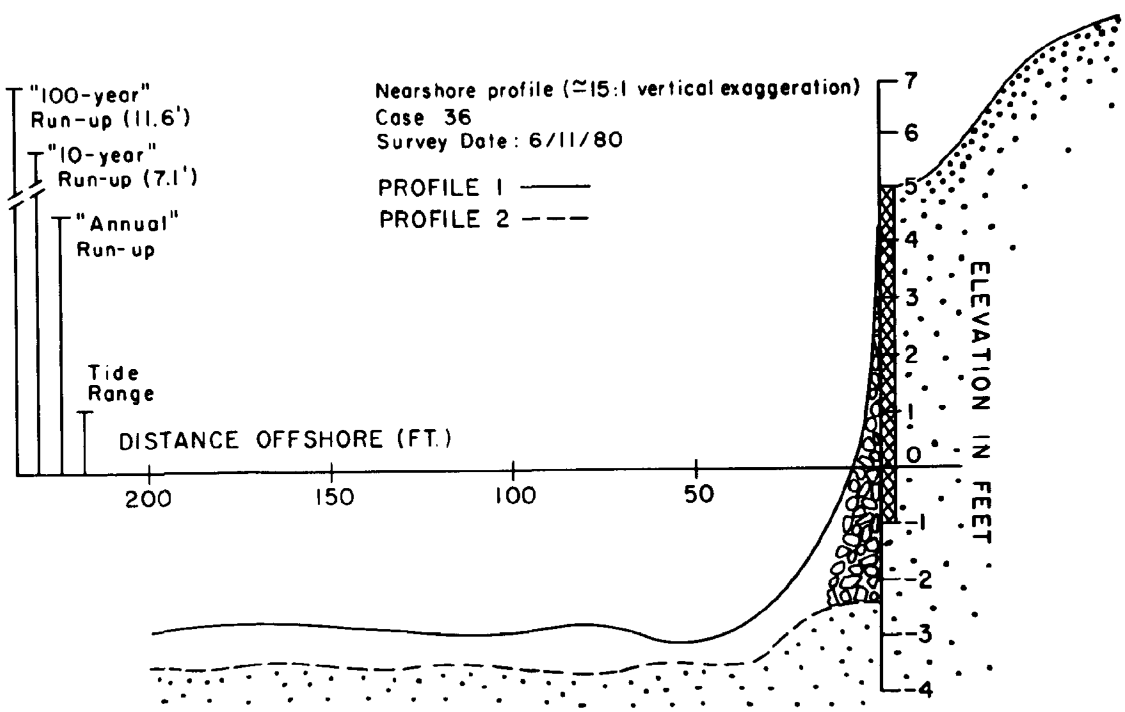
This structure protects a site composed of high sloping bank. The historical rate of erosion at the site was 8.5 ft./yr. from 1846-1942.

Structure was completed in 1974 at a cost of $56.33 per foot for the revetment placed in front of an existing timber bulkhead. At the same time, groins were completed at a cost of $53.57 per foot.

Timber bulkhead is fronted by revetment on filter cloth with 2 stone groins each 68 feet long. The rip-rap section is composed of 360-1200 lbs. stone on a 1:1.5 slope. A splashover apron consisting of boards placed on end was added 4 feet behind the bulkheading at some time after the revetment project and groins were completed.

These structures are in generally good condition. No beach has accumulated except for a small pocket of sand at the base of each groin. Profiles taken alongshore from the site show the nearshore bottom in front of the structure is relatively shallow. This suggests that the addition of rip-rap and groins reduced wave reflections off the bulkheading, and has resulted in some accumulation of sand offshore at the time of the site visit.

Ponded water on the fastland behind the bulkheading is a problem which needs correction. According to talks with local residents, the primary cause of damage to the structure has been winter ice, which displaces the stones in the rip-rap section.



CASE 36 TIMBER BULKHEAD WITH STONE REVETMENT ON KENT ISLAND

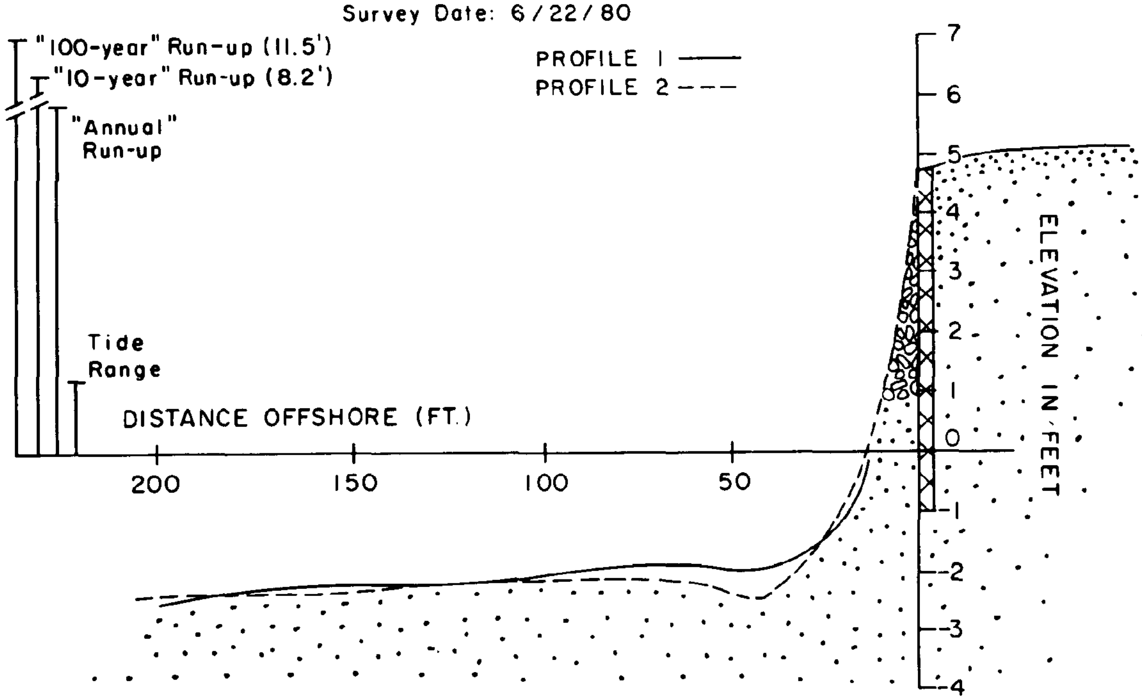
CASE 37 EXPERIMENTAL STRUCTURE ON TILGHMAN ISLAND

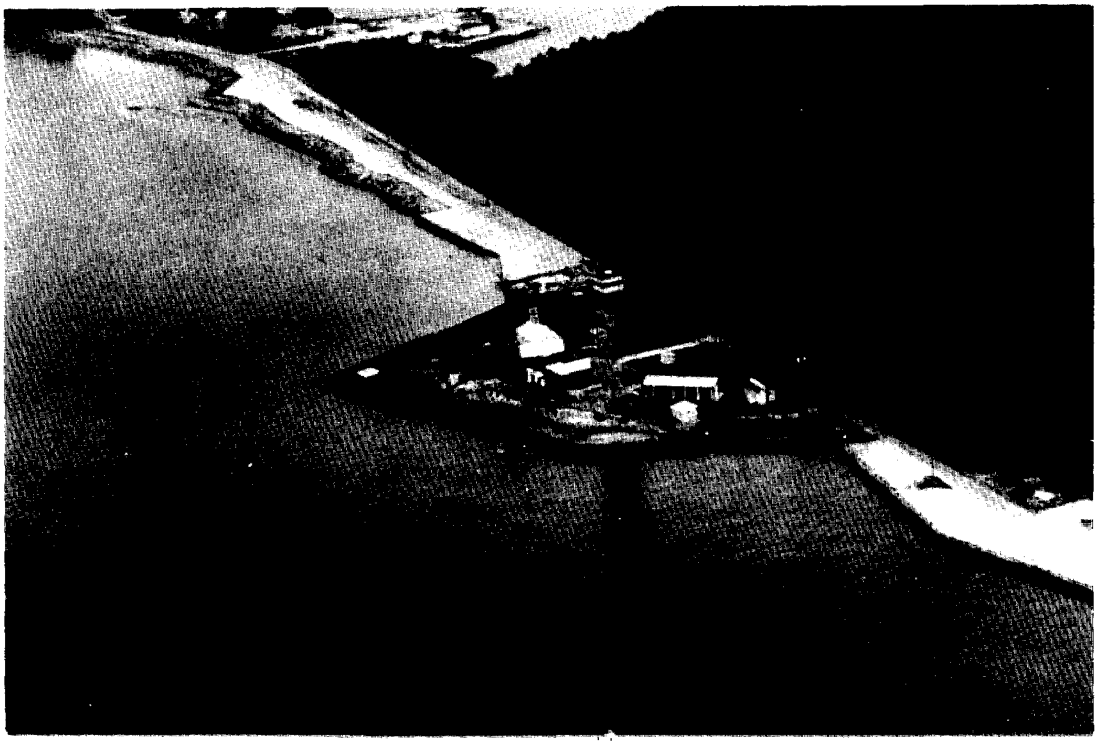
This site is composed of low fastland fronting a road. The historical rate of erosion at the site was 10.5 ft./yr. from 1847-1942.

Several experimental erosion-control structures have been installed at the site. In 1965, a sloping (2:1) wall composed of interlocking concrete block was installed along with rip-rap, and a fabriform section. These eventually failed. Presently, there is an old timber bulkhead fronted by rip-rap and a new stone revetment. A 20 foot-wide splashover apron was also installed at the site containing the stone revetment.

These structures are in good condition. There is a very small amount of sand in front of both structures. Along the shoreline to the north, the banks are severely eroding, and many fallen trees are laying in the water.

Nearshore profile (≃17:1 vertical exaggeration)
Case 37
Survey Date: 6/22/80





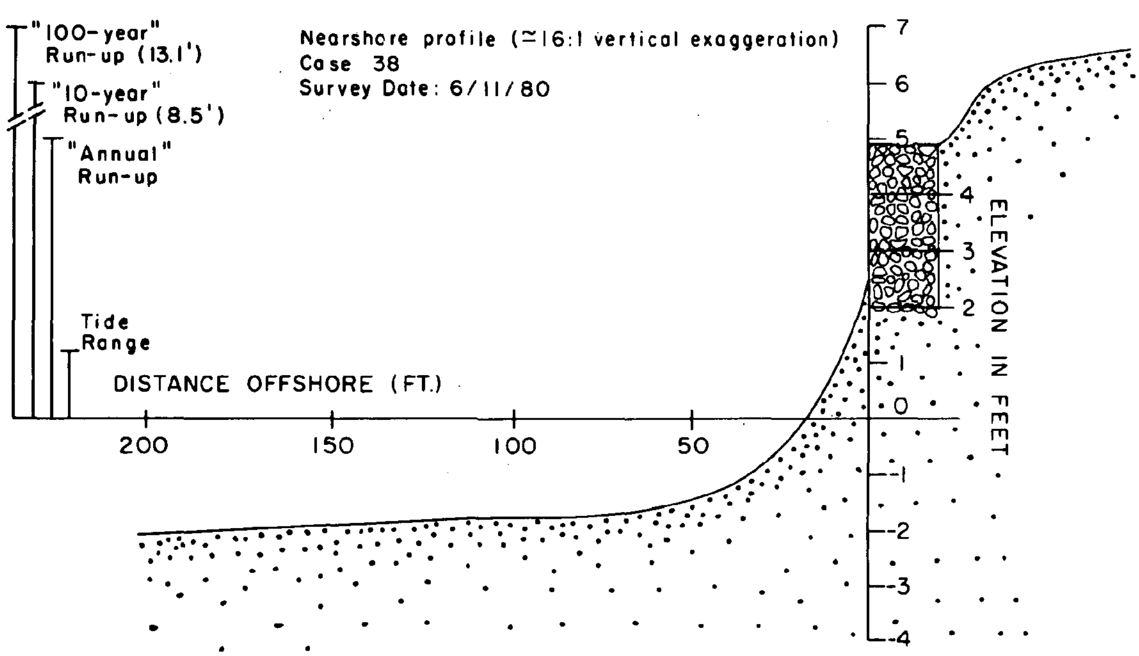
CASE 37 EXPERIMENTAL STRUCTURE ON TILGHMAN ISLAND

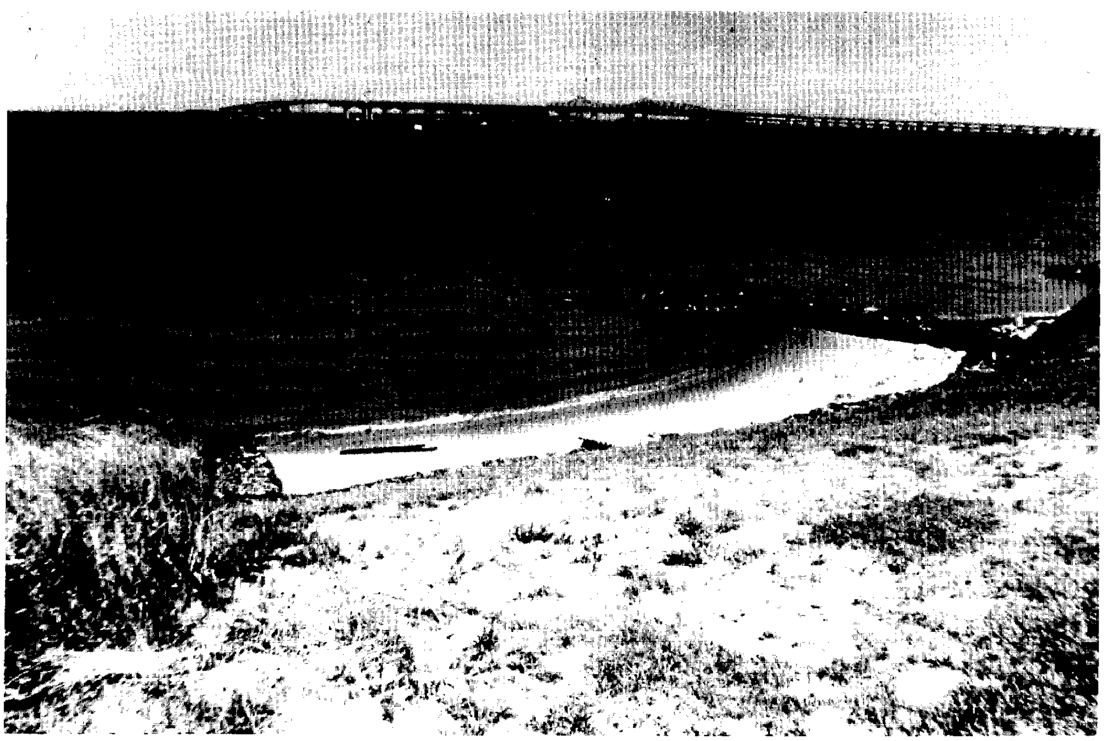
CASE 38 GABIONS ON KENT ISLAND

This site is composed of low sloping bank fronted by a sandy beach and berm. A small headland exists alongshore to the north. The historical rate of erosion at the site was 2 ft./yr. from 1844-1942.

The date and cost of the structure are not known. Gabion structure consists of a shore-parallel section and a hooked groin-breakwater section at the north end alongshore. The structure is about 5 years old. The gabions are stacked two-high.

This structure is in generally good condition. At the time of the site visit, the structure was holding a nice sandy beach extremely well. However, there is some evidence that the baskets holding the stones are deteriorating due perhaps to abrasion and corrosion.

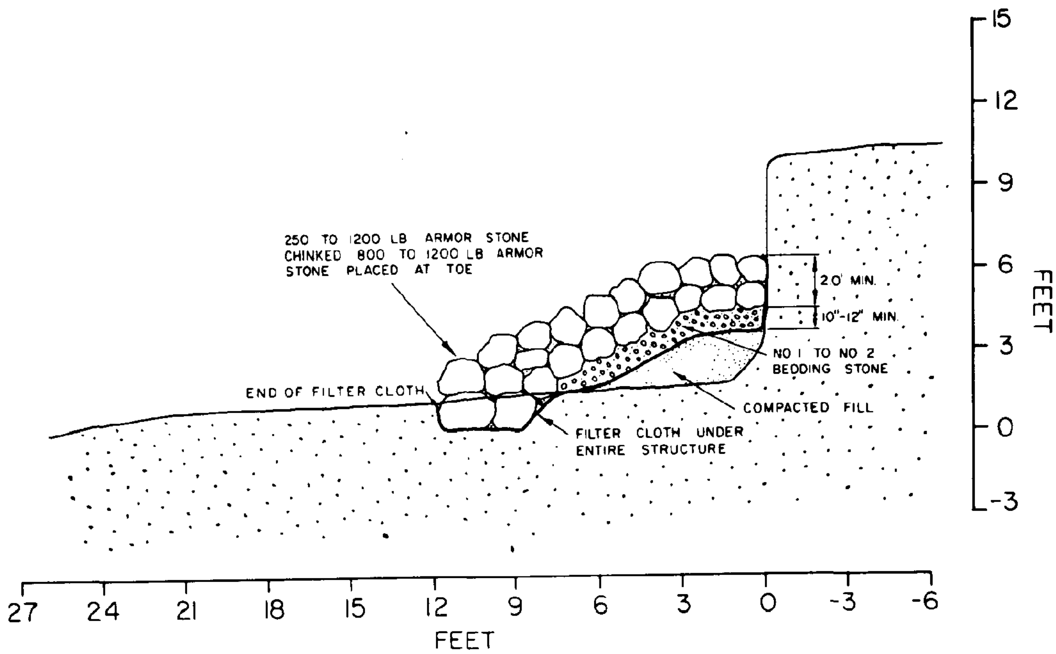


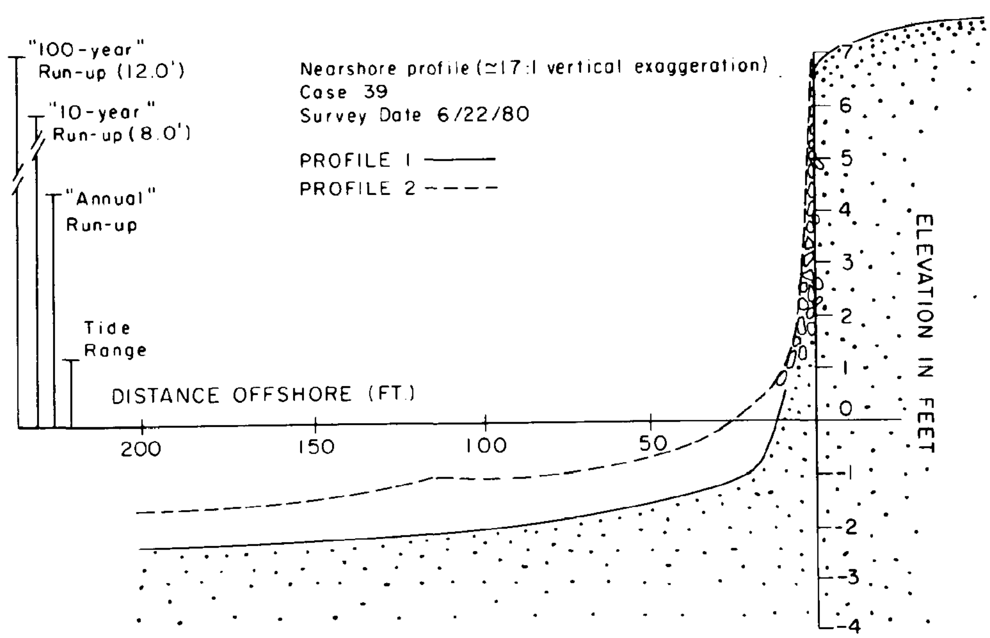


CASE 38 GABIONS ON KENT ISLAND

CASE 39 CONCRETE BULKHEAD/STONE REVETMENT AT WADES POINT

Stone revetment was completed in 1975 at a cost of $40.22/ft. The historic rate of erosion at the site was about 3 ft./yr. from 1847-1942. Stone revetment consists of an armor layer of 250-1200 lbs. stone in a 2 ft.-thick layer. A 10 in.-thick bedding layer was installed under the armor layer. Filter material was used below the bedding layer. A 3 ft.-wide splash apron was also built. This structure is holding up quite well. Alongshore, an unprotected section of shoreline is eroding rapidly. Offshore, there are 8 groins which are submerged. These are approximately 45 years old and were once attached to the shore. They presently serve no useful purpose.





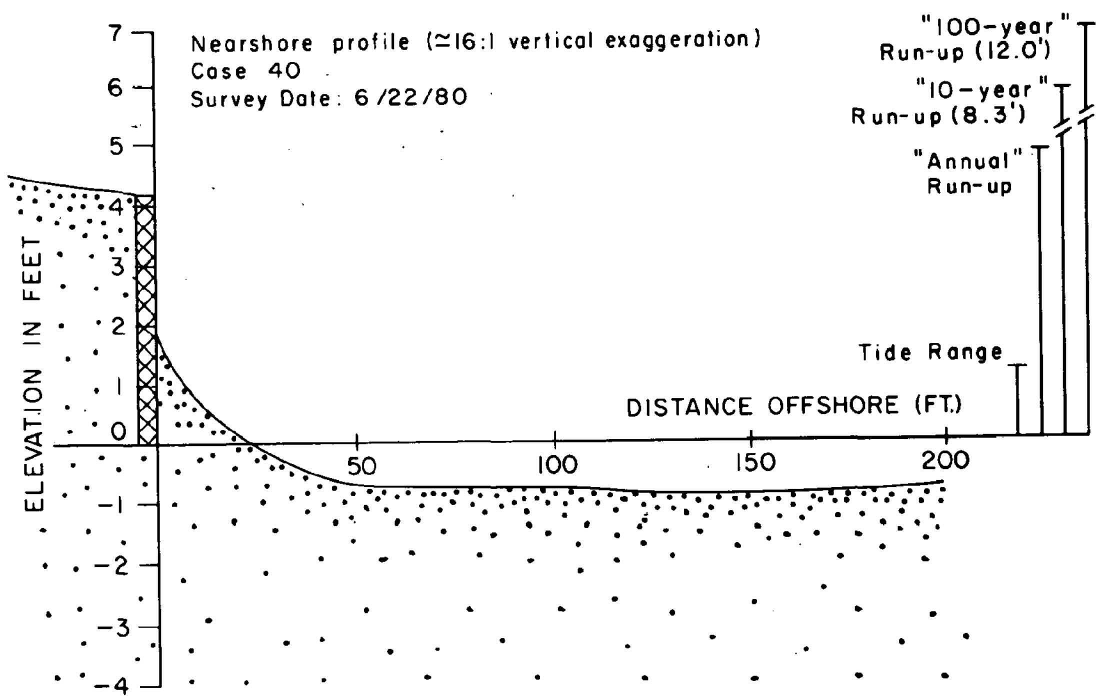
CASE 39 CONCRETE BULKHEAD/STONE REVETMENT
AT WADES POINT

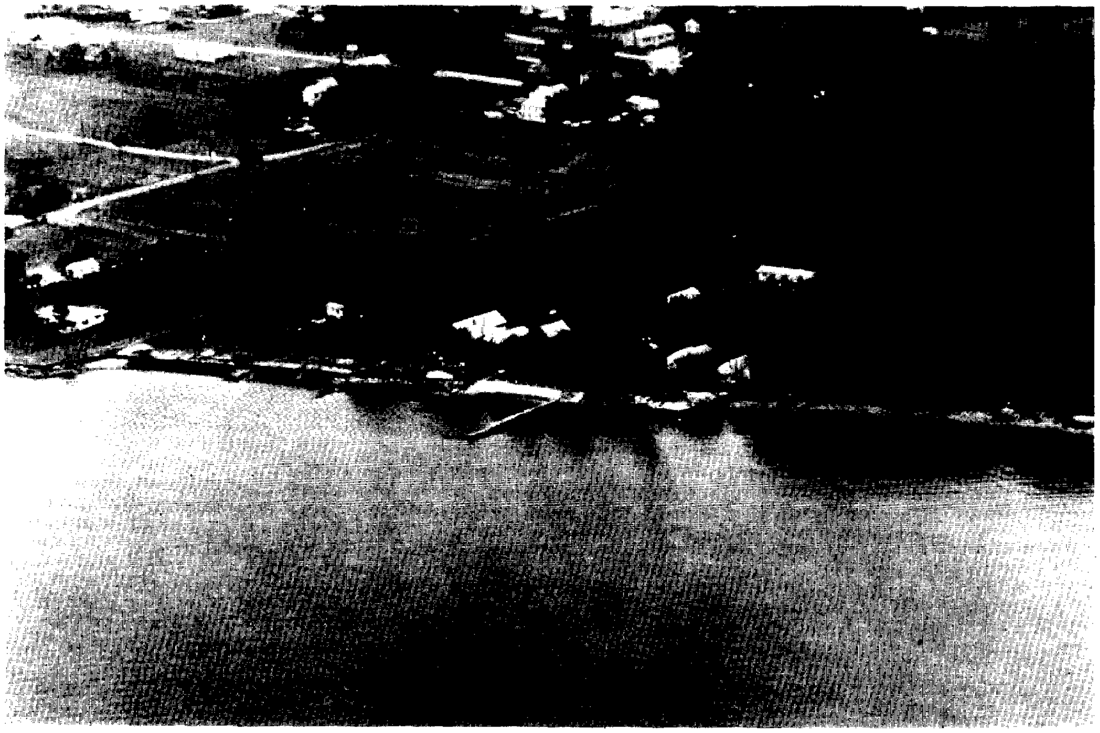
CASE 40 CONCRETE PIPES ON TILGHMAN ISLAND

This site is composed of a low grassy bank. The historical rate of erosion at the site was about 10 ft./yr. from 1847-1942. The date and cost of the structure are not known.

Short sections of concrete pipe are laid horizontally parallel to the shore. Recently, a sloping smooth concrete revetment has been placed against the fastland, and above the pipe. Alongshore, concrete pipes have also been placed seaward of a timber bulkhead. A pier constructed of concrete pipe also extends offshore.

This structure is in generally good condition. At the time of the site visit, there was a sand beach present. Alongshore to the south, trees are falling off the eroding bank. Alongshore in the other direction, the concrete pipes are installed in front of the bulkhead, and seem to be working fairly well in preventing erosion.





CASE 40 CONCRETE PIPES ON TILGHMAN ISLAND

H. Summary

It is clear from this assortment of case studies that the different types of shorelines on the northern Chesapeake Bay in general can be well-protected by erosion-control structures. But, the individual characteristics of each structure will depend on the particular physical setting. These design features are discussed in greater detail in the next chapter.

From the examples which were included in the case studies, it is evident that groins were not trapping substantial quantities of sand in some areas to form beaches; but, most of the cases of revetments and bulkheads were providing reasonable protection against wave attack for low-level storm conditions. The major deficiencies which were noted most of the time during the site visits were:

(1) overtopping of structures by waves, with minor but bothersome erosion of the fastland, and salt damage to vegetation behind the structures.

(2) lack of periodic maintenance and repair of damage to structures from storms or winter ice.

The types of maintenance which need to be performed on structures is discussed in the next chapter. It needs to be pointed out that for cases where an erosion-control structure is performing satisfactorily but where no additional vegetative measures have been employed on bluffs or high banks, there can be continuing bank instability and bluff collapse due to normal weathering and groundwater seepage (Palmer, 1973).

For either design or maintenance of any erosion-control structure, serious consideration should be given to the combination of maximum tides and waves (run-up) which can be expected in the lifetime of structures on the northern Chesapeake Bay shoreline. The drawings of the nearshore profile at each case study site show either the approximate storm surge or wave run-up conditions (storm surge plus wave height) which are predicted for storms with different recurrence intervals. The methods which were used to derive these predicted storm conditions are explained in Chapter V.

It is important to note that these elevations for either "annual" and "100-year" surge or wave run-up will <u>not</u> occur exactly once every year, or once every 100 years, respectively. If a storm surge (or storm wave) is considered to have a return period of "T_R" years, where "T_R" is 100 for the "100-year" storm surge, then the probability that this surge, or a higher one, would occur in any one year is $(1/T_R)$. The probability that it does not occur is $(1 - 1/T_R)$. (Note that the sum of the probabilities must equal one; that is, the surge either does or does not occur.) The probability that it does not occur for "n" successive year is $(1 - 1/TR)$. Finally, the probability that the storm surge <u>will</u> occur in "N" years is

$$1 - (1 - 1/TR)^N$$

Table (2.2) shows the probabilities that the "T_R" surge elevation will be equalled or exceeded for various "N" years.

TABLE 2.2
Table of Exceedance Probabilities

STORM RETURN PERIOD	"N"						
T	1	5	10	20	50	100	200
10	0.10	0.4095	0.6513	0.8784	0.9948	.99997	1.000
20	0.05	0.2262	0.4012	0.6415	0.9231	.9941	1.000
50	0.02	0.0961	0.1829	0.3324	0.6358	.8574	.9824
100	0.01	0.0490	0.0956	0.1821	0.3950	.6340	.8660

As an example, for a structure which has a design life of 10 years, the probability that the "100-year" surge will occur during a 10 year period is 9.56%.

A recommendation is offered in the next chapter that all structures, at a minimum, should be designed for top elevations greater than the "annual" run-up to avoid serious overtopping damage.

CHAPTER III

DESIGNING FUTURE STRUCTURES

Robert Dean, Robert Dalrymple,

Hsiang Wang, and Robert Biggs

A. Introduction

Different types of shorelines can be well-protected by erosion control structures, as long as the individual components of each structure, such as seawall elevation or stone size, are adjusted to the particular physical setting at each individual shoreline site. Several recommendations for design features and maintenance are presented in this chapter to improve the performance of erosion control strategies along the northern Chesapeake Bay shoreline. These recommendations are derived from analysis of the forty case studies described in the previous chapter.

These recommendations include:

1) selecting the proper crest elevation of any vertical structure
2) selecting the proper stone armor weight for revetments
3) using filter material
4) providing toe protection for vertical bulkheads
5) preventing flanking of structures due to erosion alongshore

The following Chapter IV presents some reasons for selecting from among the different structure types and two alternative "non-structural" strategies for erosion control are discussed: (1) beach nourishment; and (2) vegetative planting.

B. Selecting the Proper Crest Elevation for Vertical Protective Structures

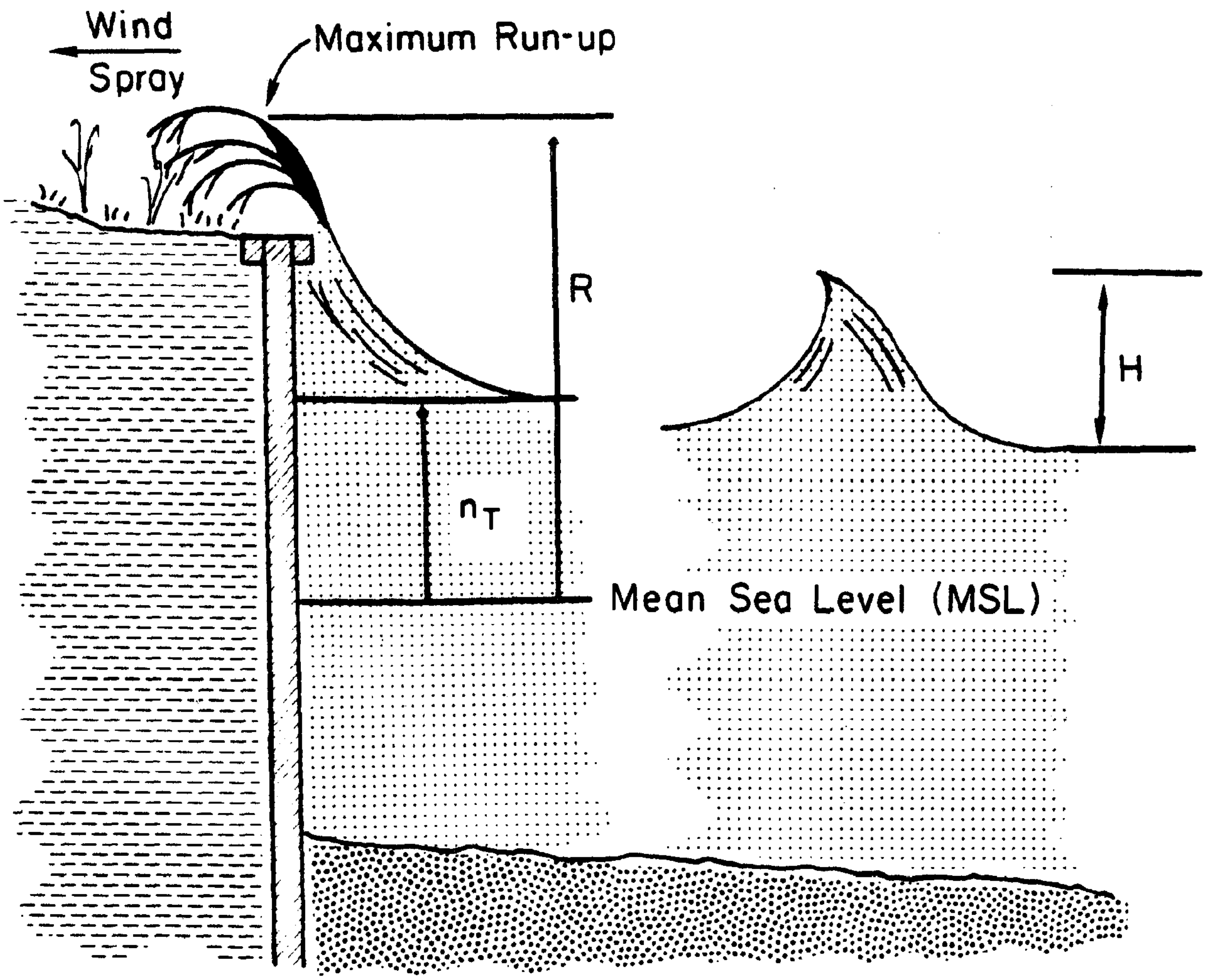
Several of the case studies described in the previous chapter showed evidence of wave splashover or overtopping of the structure. It is strongly recommended that all structures, as a minimum, should be designed for top elevations equal to or greater than the wave runup from an "annual" storm to avoid serious damage to the fastland from overtopping. The correct combination of storm-tide and wave conditions which will result in overtopping at different locations on the northern Chesapeake Bay shoreline can be derived from mathematical engineering equations and from the observations of erosion and vegetation damage at the forty case studies.

The wave run-up for vertical walls can be computed from mathematical equations which are based on small amplitude wave theory (Shore Protection Manual, 1977; Saville, 1958). The definition sketch in Figure 3.1 shows that overtopping will occur when the combination of tide plus wave run-up on the structure exceeds the maximum structure elevation. Onshore winds, of course, will often be present and will serve to transport a portion of the water exceeding the maximum elevation of the structure onto the upland properties as spray. The correct combination of storm-tide and wave characteristics to be considered for any design to prevent overtopping depends on the amount and frequency of overtopping considered "tolerable".

Opposite: Figure 3.1. Schematic diagram showing wave run-up elevation and overtopping of structures due to storm tides and waves.

Figure 3.1

OVERTOPPING OF A STRUCTURE DUE TO STORM TIDES AND WAVES



Wave Run-up From "Annual" Storm		"Annual" Storm Surge		r = 1 For Vertical Walls		Wave Height

$$"R" = n_T + ("r" \times "H")$$

Some idea of the heights of storm tides associated with storms of different return intervals is shown by plots in Figure 3.2 of height-frequency distributions from historic tidal records at the stations of Baltimore, Annapolis, and Solomons Island. These stations roughly represent three regions--upper, middle, and lower - in the northern Chesapeake Bay.

To use these data for surge predictions, the statistical significance of these data first needs to be understood. As can be seen in the graphs of Figure 3.2, the maximum storm surges with annual frequency smaller than say, 0.1 (with return period longer than 10 years), are produced by tropical storms, whereas those of return periods less than 10 years (more than 0.1) are due mainly to extra-tropical storms. The difference between these two types of storms is explained in Chapter V, where storm surges are discussed. Because the extratropical storms are far more frequent, the surge data associated with these storms are expected to be far more statistically meaningful.

Table 3.1 shows the heights "B" of 13 bulkheads included in the case studies in Chapter II, together with the predicted water depth "D" during "annual" storm conditions. This depth is equivalent to the total storm water depth plus the storm wave height. Also included in Table 3.1 is a listing of the field observations of overtopping at each site. Whenever overtopping was actually observed, a "Y" was

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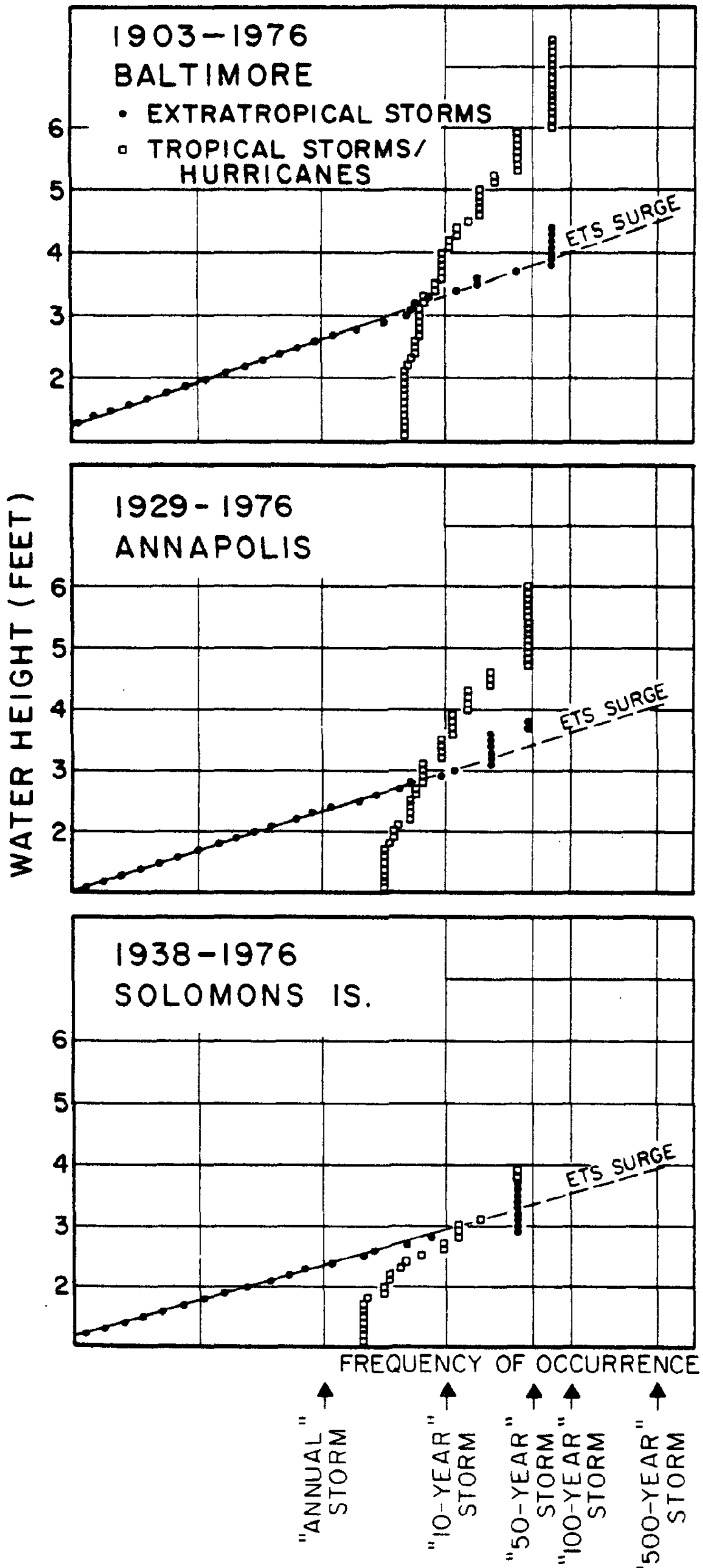
Opposite: Figure 3.2. Graphs of historic tidal data showing storm-surge heights above Mean Sea Level for storms with different return levels at three gauging stations in the northern Chesapeake Bay (from Boon et. al., 1978).

Figure 3.2


1903–1976
BALTIMORE
• EXTRATROPICAL STORMS
▫ TROPICAL STORMS/ HURRICANES
ETS SURGE
1929-1976
ANNAPOLIS
ETS SURGE
1938–1976
SOLOMONS IS.
ETS SURGE
WATER HEIGHT (FEET)
2
3
4
5
6
FREQUENCY OF OCCURRENCE
"ANNUAL" STORM
"10-YEAR" STORM
"50-YEAR" STORM
"100-YEAR" STORM
"500-YEAR" STORM

entered in the fourth column. Table 3.1 shows that only four of the 13 case studies were designed with elevations greater than the expected "annual" overtopping elevation "D". Of these four, two did not show evidence of overtopping.

Figure 3.3 shows the same results from the case studies plotted in a graphic format. For structures designed with a top elevation equal to the expected wave run-up from "annual" storms (i.e. "D" = "B"), a line was drawn (denoted 1:1) on the graph. With the exception of the Cases No. 8 and No. 35, all structures where overtopping was actually observed (i.e. where "B" was less than "D") lie below the 1:1 line, while all structures where overtopping was not noticed lie above it. Thus, a calculation of "D", which characterizes "annual" storm conditions, provided a reasonable method for deriving a design requirement which seemed to avoid overtopping at two of the structures which were among the case studies.

The equation given below for calculating "D" utilizes Mean Low Water as a datum elevation, and the terms used are different than those in the definition sketch shown in Figure 3.1. This predicted overtopping elevation "D" from "annual" storms can be computed as:

$$D = h + R. \qquad 3.1$$

Where:

"h" = <u>total</u> <u>stormwater</u> <u>depth</u> at the toe of the structure. This is determined by measuring the depth below MLW at the toe of any structure, and adding the mean tide level (which can be approximated by one-half the tidal range) plus a storm surge for an "annual" storm which can be read from the graphs in Figure 3.2.

"R" = the wave height associated with 30 mph winds (H_{30}). This can be derived from a series of atlas maps prepared for this study. (See Figure 5.12). For a rough approximation, 80% of the total storm water depth (h) can be used; however, this approximation may prove to be very conservative for sites fronted by deep water.

All structures, as a minimum, should be designed for top elevations greater than the "annual" run-up to avoid serious overtopping damage. The wave climate around the Bay is one of the most evident factors for causing shoreline erosion and for potentially damaging shoreline structures, particularly vertical protective walls. Serious consideration must be given to the combination of waves and tides (run-up) which can be expected for storms of different intensities in the lifetimes of structures on the Northern Bay. The use of predictive methods (such as Equation 3.1), together with a series of maps prepared as part of the study (see Chapter V and Appendix B) can aid in the forecast of future storm tides and wave conditions at sites of new shoreline structures.

C. Selecting the Proper Stone Armor Weight for Revetments

After reviewing the modes of failure of some of the structures, COER, Inc. recommends sloping revetments as the preferred strategy for

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Next Two Pages: Table 3.1. Calculations of wave overtopping for vertical protective structures at selected case studies of structures along the northern Chesapeake Bay.

Figure 3.3. Comparison of predicted annual overtopping of waves with field evidence of overtopping at structures along the northern Chesapeake Bay shoreline.

Table 3.1

CALCULATIONS OF WAVE OVERTOPPING FOR VERTICAL PROTECTIVE STRUCTURES

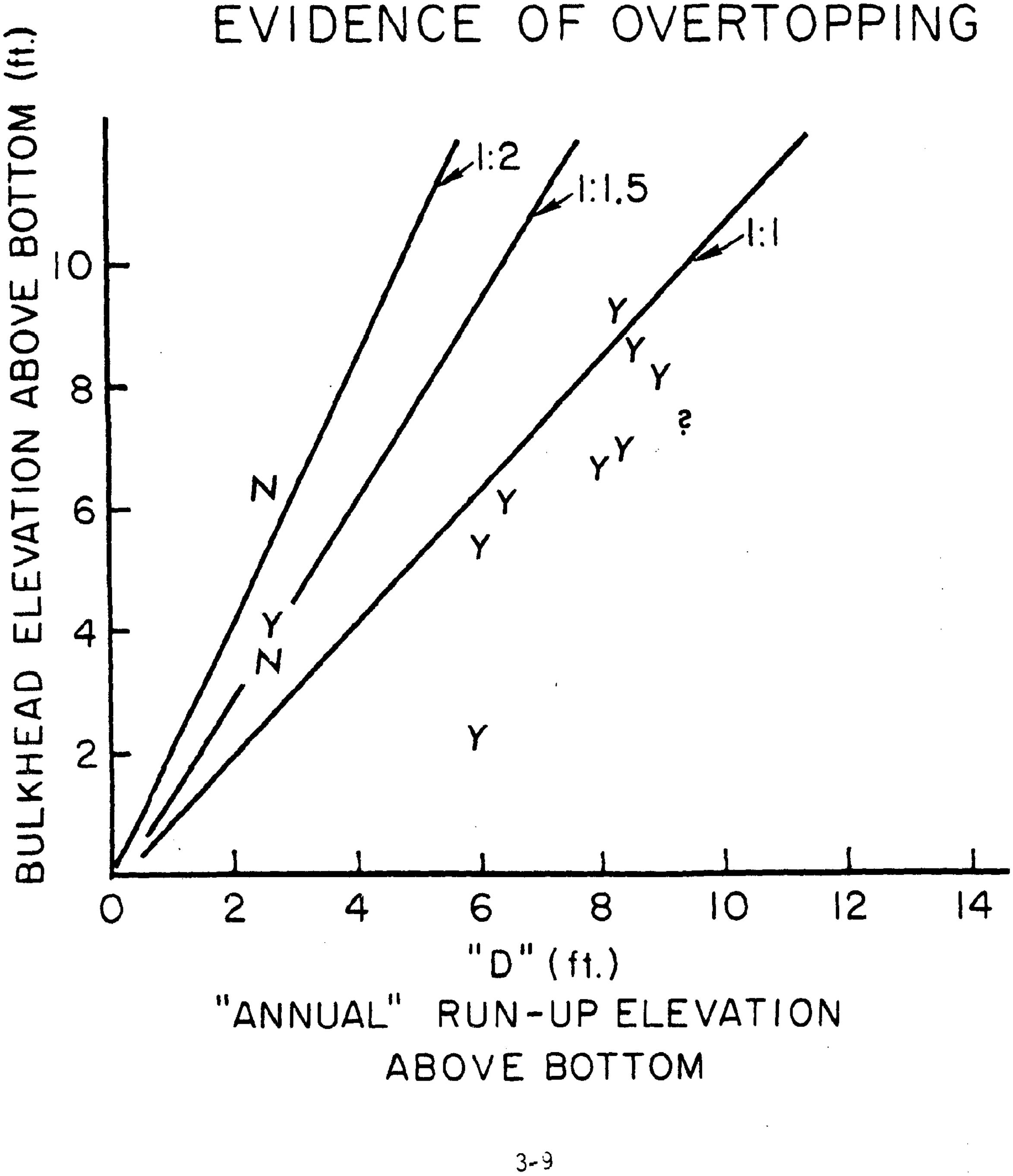
Case Study		Total Depth at Structure "D"[1] (ft.)	Bulkhead Height B[2] (ft.)	Potential Overtopping D<H	Field Observed Overtopping
15	Randle Cliffs bulkhead	9.6	6.2	Y	?
16	Dares Beach bulkhead	8.7	6.2	Y	Y
34	Tilghman Island bulkhead & revetment	12.6	11.21	Y	Y
14	Tall Timbers bulkhead	8.7	8.2	Y	Y
8	Honga River bulkhead	2.4	3.8	N	Y
26	Middle River bulkhead & revetment	2.5	6.4	N	N
9	Trippe Bay bulkhead	6.5	5.8	Y	Y
10	Choptank River bulkhead	6.2	5.1	Y	Y
35	Kent Island bulkhead	8.3	8.7	N	Y
36	Kent Island bulkhead & revetment	9.4	7.5	Y	Y
20	Hillsmere Beach bulkhead	2.5	3.6	N	N
28	Back River bulkhead	6.0	2.5	Y	Y
7	Honga River bulkhead	8.6	6.2	Y	Y

[1] Total depth at structure "D" = total storm water depth + wave height ("H_{30}")
= water depth (MLW) at toe of structure + mean tide level + annual storm surge + 30 mph wave height or 0.8 of the total storm water depth (whichever is smaller).

[2] Total elevation above bottom.

Figure 3.3

COMPARISON OF PREDICTED "ANNUAL" OVERTOPPING WITH FIELD EVIDENCE OF OVERTOPPING



many more shoreline situations in the northern Chesapeake Bay. This method for erosion control offers the following advantages:

(1) The materials used to build revetments do not degrade with time.

(2) Sloping revetments are unlikely to experience catastrophic failure. (If design conditions should be exceeded slightly during a storm, inevitably some stones may become dislodged and these can be replaced afterwards.)

(3) Wave reflection from sloping revetments is usually low; thus, less disturbance and less scour of sediments results at the toe of the structure.

(4) Rubble generally provides a better habitat for biota than the materials which are used in most other types of shore protection.

This section develops and presents recommendations related to the establishment of reasonable stone weights for structures along various portions of Chesapeake Bay.

The design median armor stone weight "W_{50}" for a particular structure is usually determined in accordance with the following equation taken from the U.S. Army Corps Shore Protection Manual (1973):

$$W_{50} = \frac{\gamma_r H^3}{K_D (S_s - 1)^3 \cot \alpha} \qquad 3.2$$

where:

H = the design wave height.

$\cot \alpha$ = where α is the angle that the structure makes with the bottom.

γ_r = the specific weight of the armor stone.

S_s = the specific gravity of the stone relative to the water in which it is placed. Generally, although not always, the specific weight of stone varies over only a narrow range, and can be approximated by $S_s = 2.6$.

K_D = the stability coefficient depending on the interlocking or frictional characteristics of the armor stones. K_D for reasonably angular rock is usually taken as 2.0 to 3.0, and for purposes here a value of 2.0 will be adopted.

In order to develop a general guideline for stone weight selection, Equation 3.2 was simplified as follows:

$$W_{50} = 20.2\ H^3/\cot \alpha \qquad 3.3$$

where:

W_{50} = the armor stone weight in pounds.

H = the design wave height in feet.

Of forty case studies described in the previous chapter, thirteen were revetment structures whose actual designs can be compared with the design that would be aasociated with storms of various return periods. The stone weights for these thirteen structures are presented in Table 3.2.

Based on a "W_{50}" stone weight, and the bathymetry at the time the profiles were collected in conjunction with this study, the design conditions are such that five of the thirteen structures have design return periods of less than one year, which means they have probably experienced the maximum wave for which they are designed more frequently than once a year. This is quite unexpected, since only one of the structures has failed and the average in-service life is four years for the four that showed no visible signs of failure. There are two other possibilities for this apparent discrepancy: (1) the calculated wave heights are too large or the actual wave heights did not persist long enough to accomplish failure, and/or (2) the definition of damage associated with the breakwater stability equation is so strict that such damage would not be evident in the field. The damage criterion relates to a displacement of 5% of the stones. Presumably, the movement would occur first among the smaller stones. If Equation 3.2 is used with the upper limit of stone weight contained in each revetment, then only the structure in Case No. 34 (stone revetment on Tilghman Island) is in a class with a return period of less than one year. This structure has been in service for approximately four years at the time of the site visits, and there was no record nor

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Opposite: Table 3.2. Characteristics of revetment structures discussed in Chapter II.

Table 3.2 Characteristics of Revetment Structures Included in Assessment

Case Identification No.	Stone Weight* Range (lbs)	w_{50}** (lbs)	cot α	Apparent Design Return Period (yrs) e = 0	Wave Heights (ft)			
					Design	Wave Heights of One-Year Return Period For Erosion, e, of		
						e = 0'	e = 1'	e = 2'
36	350-1200	650	1.5	< 1 yr	3.6	4.0	4.0	4.0
4	400-1000	630	2.0	8 yrs	4.0	3.5	4.2	5.0
33	350-1200	650	2.0	> 5 yrs	4.0	0.6	1.4	2.1
13	1400-2800	1980	2.0	50 yrs	5.8	3.9	4.6	5.4
3	450-1200	735	2.0	< 1 yr	4.2	4.4	4.6	4.6
2	350-1200	650	2.0	20 yrs	4.0	2.6	3.4	4.2
1	400-1200	690	2.0	< 1 yr	4.1	4.6	4.6	4.6
6	400-1200	690	2.0	8 yrs	4.1	3.7	4.5	5.3
11	250-1000	500	2.5	8 yrs	4.0	3.4	4.2	5.0
12	400-1000	630	2.0	6 yrs	4.0	3.0	3.0	3.0
18	30-300	95	2.0	< 1 yr	2.1	2.5	3.3	3.3
34	400-1200	690	2.0	< 1 yr	4.1	4.9	4.9	4.9
5	400-1000	630	2.0	12 yrs	4.0	3.2	4.0	4.8

*Obtained from pre-construction engineering cross-sections.

**W_{50} was obtained from the stone weight range by $w_{50} = 1.85\ w_{MIN} + w_{MAX}/1.85$. This formula is empirically based.

visual indication of revetment failure. Thus, the only possible conclusion is that the calculated wave heights are larger than occurred in nature or their duration was too short to allow failure to develop.

If additional erosion takes place at the toe of sloping revetments such that the offshore water depth increases, then larger waves may be able to travel onto the shore without breaking. In this case, the revetment may sustain damage from storms which will occur more frequently than the design storm.

To illustrate the effect of future erosion fronting a revetment structure, detailed plots have been developed for the three following structures: No. 11, No. 10 and No. 34 (Figure 3.4). Figure 3.4 shows that the structure No. 11, with no additional depth increase, is designed for a storm with a return period of approximately eight years. If an erosion of even one foot should occur, the indicated return period would be less than one year. It is informative to examine the "W_{50}" requirements for various erosion considerations and for a particular storm return period. For example, for a return period of 20 years, the "W_{50}" values are presented in Table 3.3. It is clear that the depths fronting the Case No. 11 structure reduce substantially the wave heights that can impinge on the revetment. If sufficient erosion would occur such that all waves would not break prior to impinging on the structure, the required stone weights would increase six-fold over that associated with "no additional erosion".

The site for case #10 is relatively sheltered and thus the maximum waves are much smaller than at the site of case #11 . Thus,

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<u>Opposite</u>: Figure 3.4. Required armor weight for storms of varying return period at three shoreline sites in northern Chesapeake Bay.

Figure 3.4

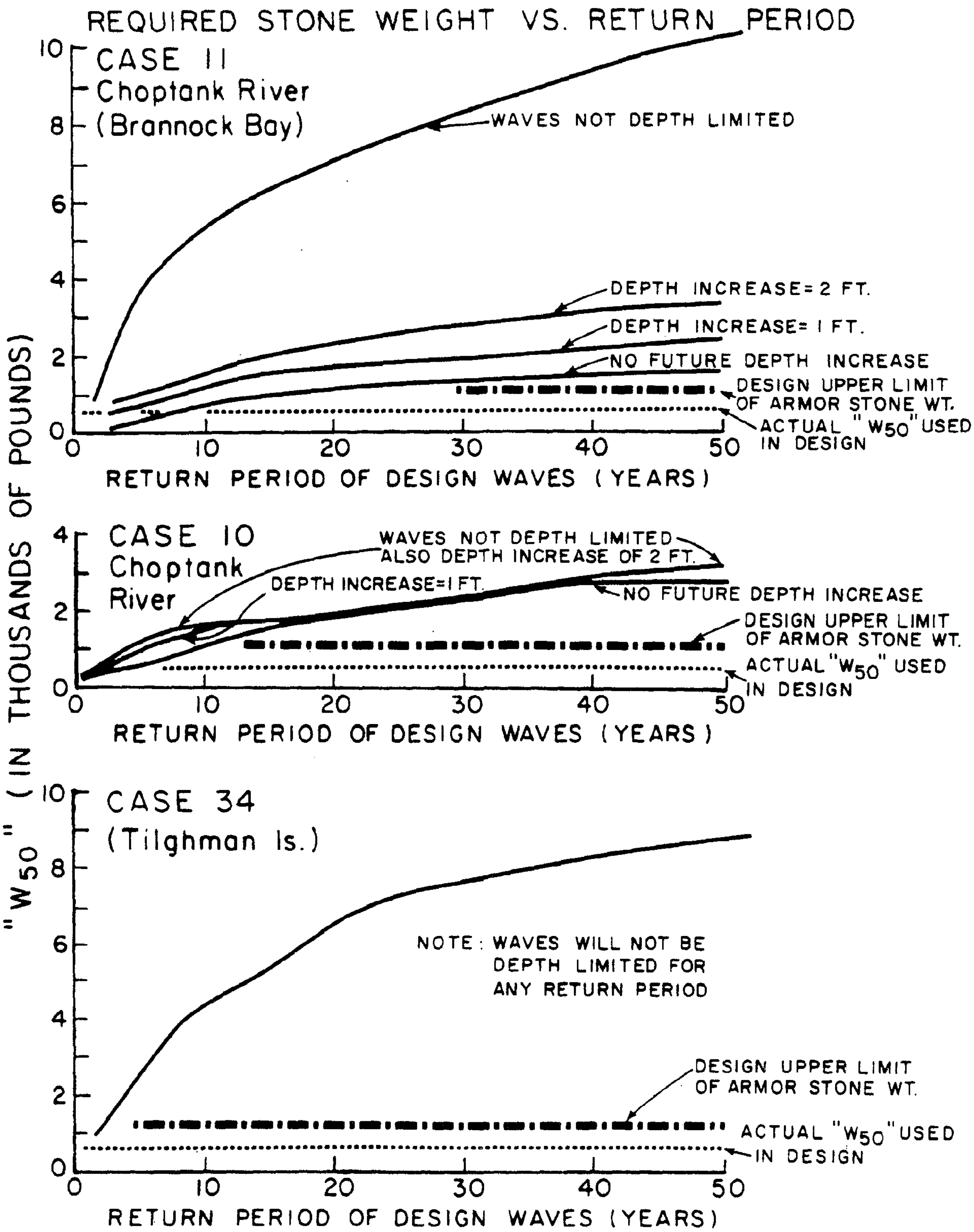
REQUIRED STONE WEIGHT VS. RETURN PERIOD
CASE 11
Choptank River
(Brannock Bay)
WAVES NOT DEPTH LIMITED
DEPTH INCREASE= 2 FT.
DEPTH INCREASE= 1 FT.
NO FUTURE DEPTH INCREASE
DESIGN UPPER LIMIT OF ARMOR STONE WT.
ACTUAL "W50" USED IN DESIGN
RETURN PERIOD OF DESIGN WAVES (YEARS)
CASE 10
Choptank River
WAVES NOT DEPTH LIMITED ALSO DEPTH INCREASE OF 2 FT.
DEPTH INCREASE=1 FT.
NO FUTURE DEPTH INCREASE
DESIGN UPPER LIMIT OF ARMOR STONE WT.
ACTUAL "W50" USED IN DESIGN
RETURN PERIOD OF DESIGN WAVES (YEARS)
CASE 34
(Tilghman Is.)
NOTE: WAVES WILL NOT BE DEPTH LIMITED FOR ANY RETURN PERIOD
DESIGN UPPER LIMIT OF ARMOR STONE WT.
ACTUAL "W50" USED IN DESIGN
RETURN PERIOD OF DESIGN WAVES (YEARS)
"W50" (IN THOUSANDS OF POUNDS)

Table 3.3

"W_{50}" Requirements For Three Revetment Structures For Various Erosion Scenarios with a Twenty-Year Return Period Design

Offshore Erosion Scenario	Median Armor Stone Weight (lbs.) "W_{50}" for Structure at: Case No. 11	Case No. 10	Case No. 34
No Additional Erosion	1200	1600	6700
1 ft Additional Erosion	1600	1640	6700
2 ft Additional Erosion	2400	1640	6700
Waves Not Depth-Limited	7200	1640	6700

any additional offshore depth increases can cause only negligible increases in the required "W_{50}" values over the "no additional erosion" scenario.

Finally, the No. 34 structure is fronted by water of substantial depth (= 6 ft MSL), and none of the waves (up to a return period of 50 years) are depth-limited. Therefore, the required weight "W_{50}" is the same for all depth scenarios.

A favorable factor relating to stone revetments is the nature of revetment failures. When rubble revetments fail, generally they do not fail catastrophically, but fail through dislodgement of several stones. Moreover, several hours may be required to reach a near-equilibrium damage level for a particular design storm. Of course if wave heights occur that are much greater than the design wave height, there could be complete and rapid failure of the structure. Regardless of the degree of failure, the individual stones do not suffer damage and, if they can be recovered, they can be replaced in a partially damaged revetment or used in a structure designed for a larger wave height. The recovered stones could be augmented by larger stones to increase the "W_{50}" value or the "W_{50}" value could be kept the same and a milder revetment slope used. The engineering cross-sections for many of the revetments described in Chapter II have slopes of about 2:1.

In summary, for the revetted structures, it has not been possible to determine the exact cause of the apparent discrepancy between the calculated return periods associated with the various designs and the performance history. In view of the indicated effects associated with future depth increases, the following are recommended:

(1) A continuing program of monitoring revetment structures. Monitoring should occur periodically on an annual basis and after severe storms.

(2) The wave height calculation procedure used in this study should be evaluated against high quality wave measurements in Chesapeake Bay. At least five wave gauge installations at selected locations should be installed for a one-or two-year duration.

(3) In the design of revetment structures, it is recommended that consideration be given to designing for a larger wave height and storm tide combination. In particular, until improved procedures are developed, a design wave height

and storm tide equal to 1.5 times the "one-year" wave height "H_1" and storm tide "η_T" are recommended for consideration. Calculations show that this would result in an approximate average design return period of 10-11 years.

(4) Minimum stone weight "W_{50}" of 650 lbs is recommended. (This implies a range of stone weights of 350-1200 lbs.) If the water depth fronting a particular structure is still sufficiently shallow (after reasonable consideration for future erosion) so as to limit the wave height, the design wave can be reduced accordingly. To provide an indication of the effect of this recommendation for the thirteen structures examined, the required "W_{50}" values would range from 450 to 8500 lbs. considering an erosion of 1 foot. The average ratio of recommended to actual stone weights for the individual structures is 1.0 to 12.3, with an overall average ratio of 3.54. Although this represents a substantial increase in the weights of the individual stone sizes, the corresponding increases in volumes in on the order of 50%, and the associated cost increase is expected to be 60% to 70%, which would be approximately 30% greater than the cost of timber bulkheads.

Of the forty case studies, adequate cost data were available for thirty-three of the structures. If the cost figures are adjusted to 1980 levels by using the indexes and method presented from the Engineering News Record in Table 2.1, then the ratio of the most costly (stone revetment - $214.57 per foot) to the least costly (stone groin - $26.35 per foot) is

8:1. The average adjusted cost of stone revetment ($90.71 per foot) is 30% lower than the average adjusted cost of timber bulkhead ($127.89 per foot). The average adjusted cost of the two aluminum bulkheads in the case studies is $88.11 per foot and the average adjusted cost of the 8 timber and stone groins is $92.79 per foot. It is important to note that the indexes presented in Table 2.1 are for general building trades and may not indicate the true inflation rate for marine construction.

The costs of structures are also reflected in actual 1980 bid prices for shore erosion-control projects built by the Department of Natural Resources Shore Erosion Control Program.

Table 3.4

Average Cost Per Unit Length of Various Types of Structures (1980 Dollars)

Type of Structure	Number of Structures	Average Cost Per Foot	Range of Costs Per Foot
stone revetments	18	$124.24	$101.00 - $219.18
aluminum bulkhead	2	$141.19	$135.47 - $199.89
timber bulkhead	15	$211.54	$161.33 - $328.47
Total Cost / Total Footage	$2,213,183.60 / 15,383.4	= $143.87 average cost per foot.	

D. Use of Filter Cloth in Construction

The design purpose of filter cloth is to prevent the leaching or washing out of sediment fines from behind or under the structure with the possible consequence of at least partial failure of the structure due to void creation and slumping. Although the life of filter cloth is not well established, its use is certainly recommended. In some vertical structures which were among the case studies, it was clear that leaching had occurred through the seams of adjacent sheetpile and this could have been prevented by use of high quality filter cloth.

E. Toe Protection

There are a number of qualitative advantages to be gained by providing toe protection for vertical bulkheads. Toe protection, usually takes the form of rubble, and reduces run-up, overtopping, and toe scour during storms, as well as provides a better habitat for biota. The installation of rubble toe protection should include filter cloth and perhaps a bedding of small stone to reduce the possibility of rupture of the filter cloth. Ideally, the rubble should extend to an elevation such that waves will break on the rubble during storms. In many places, the associated cost may approach a substantial percentage of that for a revetment, making both a vertical bulkhead and sloping revetment difficult to justify on economic terms.

F. Provision of Return Walls to Prevent Structure Flanking (Figure 3.5)

If a section of shoreline is protected by continuous structures of high integrity and good design, then there will be no possibility for erosion of upland property and/or erosion than can adversely affect the structure. However, if only a short shoreline segment is protected by a structure, then the effects of continued erosion of the

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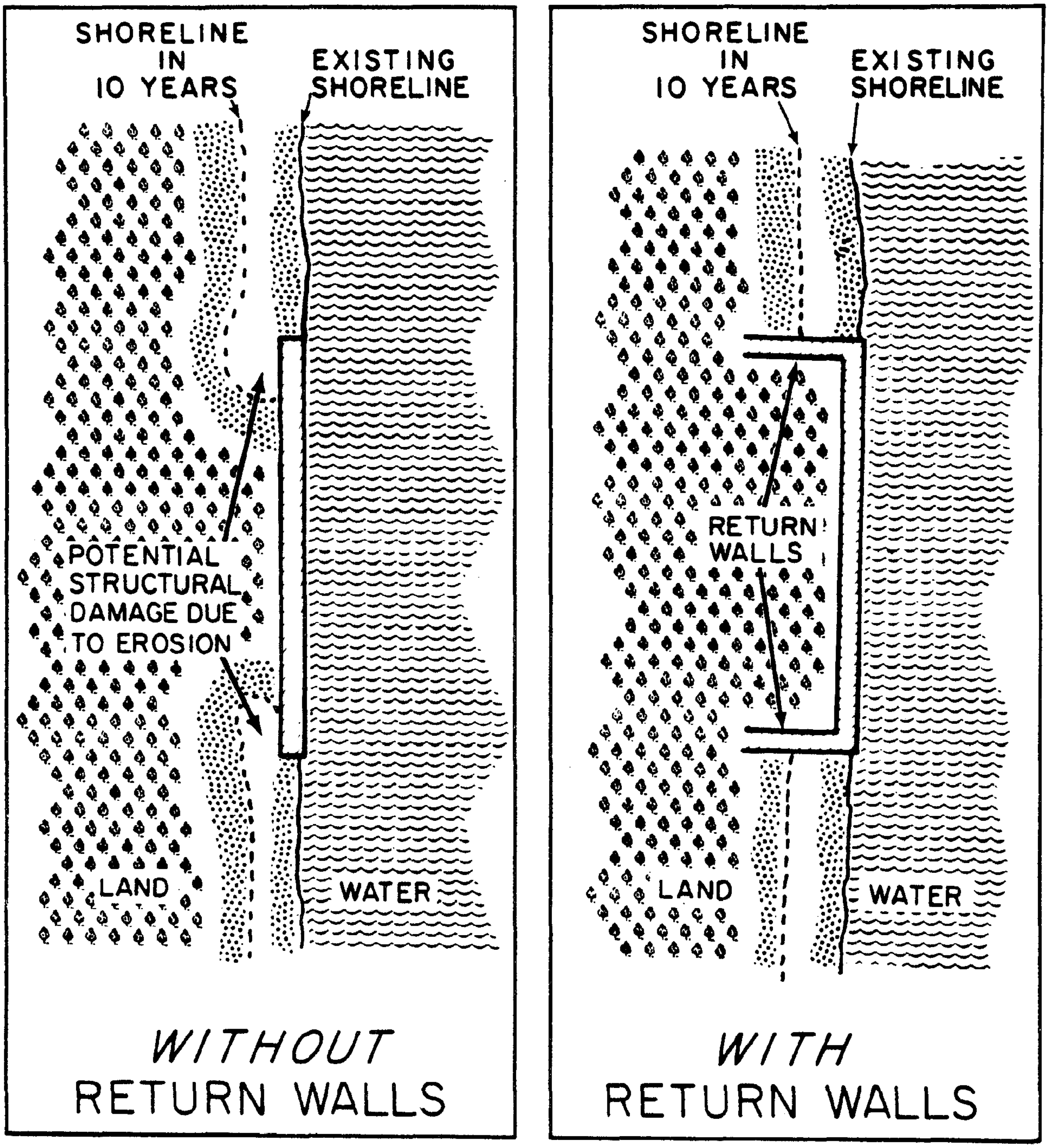
Opposite: Figure 3.5. Role of return walls at the ends of shoreline protection structures.

Figure 3.5

RETURN WALLS PREVENT FLANKING AT THE ENDS OF VERTICAL PROTECTIVE STRUCTURES



adjacent shoreline should be recognized. At several of the structures examined, this process had occurred to a degree that either flanking and damage of the structure had occurred or was reasonably imminent. To prevent flanking, return walls should be provided for a distance consistent with the erosion rate and design life of the structure (see Figure 3.5).

G. Maintenance of Structures

Periodic maintenance of structures is necessary due to annual storm and winter damage and the possible effects of flanking and offshore profile deepening. The maintenance varies with the structural type; but annual inspections should be made by the property owners. For stone revetments, the replacement of stones which have been dislodged is necessary; timber bulkheads need to be backfilled if there has been a loss of upland material, and broken sheetpile should be replaced as necessary. Gabion baskets should be inspected for corrosion failure of the wire (the plastic packet can fail due to improper handling during construction or abrasion by stones inside the baskets). Baskets should be replaced as necessary, as waves will empty failed baskets. Asbestos cement and aluminum bulkheads should be inspected for sheetpile failure due to active earth pressure or debris impact and for loss of backfill. For all structural types not contiguous to other structures, lengthening of flanking walls may be necessary every few years. Through periodic monitoring and required maintenance, a substantially greater percentage of coastal structures will perform effectively over their design life.

CHAPTER IV

Discussion

Robert Dean, Robert Dalrymple,
Hsiang Wang, and Robert Biggs

A. Summary of Observations

Although there are examples of structures along the Chesapeake Bay shoreline which definitely have not been effective in preventing beach erosion, it is very clear that structures can be designed and installed to prevent erosion of upland property without having an adverse effect on the adjacent shorelines. One of the main differences between the Chesapeake Bay shorelines and oceanic shorelines is that the material eroded along the Chesapeake Bay shoreline is predominantly very fine sediment (less than 60 microns). The later discussion of this chapter will show that the average percentage of eroded material that is of beach sand size in the bluffs and banks on the northern Bay is on the order of 10%. Thus erosion of the cliffs and bluffs does not result in significant quantities of beach sand, and any benefit that can be ascribed to allowing shorelines to erode to maintain beaches is relatively small. In fact, rational arguments can be advanced that those unprotected shoreline segments bounded by stabilized property on both ends will experience a long-term reduction in erosion forces.

The forty sites which were included in this study are protected by a variety of coastal structures: bulkheads (14 sites have timber bulkheads, 4-concrete, 3-aluminum, 1-asbestos cement), stone revetments at 16 sites, gabions at 4 sites, groins at 15 sites, well rings at 1 site, and concrete pipes at 1 site. The sites were located in eight of the Bay counties: 12 sites are in Dorchester County, 11 in Anne Arundel, 4 in Queen Annes, 4 in Talbot, 3 in Calvert and 2 sites each in St. Marys, Baltimore and Kent Counties. No sites were selected in Cecil, Harford or Somerset Counties.

The purpose of the structures at the forty sites has been to stop shoreline recession. In a majority of the cases, this has been achieved, despite the range of coastal types and structures that appear in the Bay. The use of well-designed and constructed timber bulkheads can be expected to prevent upland erosion due to wave attack regardless of the coastal features of the area. Even at Randle Cliff, where 100 ft.-high cliffs are eroding, the experience of the U.S. Naval Research laboratory has shown that (with the exception of weathering due to rain and the freeze-thaw cycle) cliff erosion can be halted.

Of the forty sites visited, five showed substantial deterioration. This deterioration is believed to have been preventable through a more proper design or maintenance and recognition of the dominant natural forces present along the shore. The effectiveness of the remaining structures is really quite remarkable when it is considered that certain portions of the shoreline of Chesapeake Bay are eroding historically at rates of 10 or more feet per year. However, in general, the wave climate is reasonably moderate and thus it is not necessary to design the structures for extremely large waves. This is reflected in the reasonable costs of the structures. Where available, the structure cost per unit length has been determined and adjusted to 1980 levels (see Chapter III, p. 3-18 and 3-19). The unit cost was found to be on the order of $150/ft.

The only <u>general</u> deficiencies occurring in a substantial percentage of the structures that were visited were: (1) the occurrence of structure overtopping with generally minor but bothersome erosion of the upland and salt damage to the vegetation behind the structure, and (2) lack of maintenance. Recognition of the combination of maximum tides and waves occurring within the various portions of the bay would minimize this overtopping problem. As far as maintenance is concerned, structures require periodic monitoring and repair for damage from winter conditions or storms.

B. General Design Recommendations

As a result of some design deficiencies noted in the field investigations, several recommendations related to design features and maintenance were discussed in Chapter III. These include: (1) the crest elevation of the structure, (2) stone (armor) weight for revetments, (3) general recommendations associated with use of filter material, (4) prevention of structure flanking, and (5) maintenance of structures. All of these recommendations are discussed at length in Chapter III.

C. Selection of Shoreline Protection Type

Each of the following structures appears to perform its design function adequately: timber, concrete, aluminum and asbestos bulkheads, stone revetments and gabions. At the stage when a shorefront property owner is selecting a structure type, the results of this study suggest that serious consideration be given to sloping stone revetments for the reasons noted below.

First, there are relatively small cost differentials (per foot of shoreline) between the stone revetments and the least costly structures. The recommended increases in design for the proper wave height and storm tide made in Chapter III should result in the costs of timber bulkheads and stone revetments being within 30% of each other. Second, all of the structural materials except rock have a limited life. In the case of well-treated timber, this life may be fairly long (say 20-30 years); however, it is still limited.

In comparing sloping stone revetment structures with more rigid structural types (including interlocking revetment), it is clear that the latter can fail catastrophically if the design conditions are exceeded or if other characteristics (such as bay bottom level) change. If such an event should occur and the rigid structure were to fail, there could be little of salvage value and, in fact, costs could be associated with the clearing of structural debris. Finally, an erosional loss of upland property can occur, even during the same storm which causes the structural failure, and in the time span prior to the installation of a new structure.

The sloping stone revetments included in this study form a sample which is too small to address the long-term performance of sloping revetments, at least under past design practices and future erosion scenarios. But the inherent advantages of this structural type merit the strong consideration of sloping stone revetments for future installations.

D. Consideration of Groins

Groins can affect nearshore sediment transport processes in a number of ways. Probably the most effective way to consider their performance is that, properly designed, they serve as a template to the beach so that a single groin will collect sand on the up-drift side to a design elevation, and then the sand either flows over or around the groin. However, groins are only effective when there is a suitable amount of beach sand available in the shoreline system. An alternate approach is to fill the groin pockets initially with suitable-sized sand, and then replenish the fill as needed.

If groins are too "severe," that is too long and/or too high, they can actually be counter-productive by inducing rip currents which can cause sand to be jetted offshore resulting in a down-drift area deficient of sediment. It should be noted that no groin-related rip currents were observed in Chesapeake Bay during this study and that the conditions under which groins will cause rip currents are not well understood.

If two or more groins are present, forming one or several groin compartments, the groins tend to collect sand until they reach their retention capacity. The sediment impounded is obtained from that moving along the shoreline, and this represents a loss of material to the adjacent shoreline. It follows that if one or a series of groins is not to cause an erosion of the adjacent shoreline, the groins should be filled to their capacity with quality sand (beach sand size) when installed.

Groins should be sandtight if they are to retain sand. The sand-tight property results from installation of a graded stone cross-section such that only small interstices occur within the groin matrix.

Some groins are not necessarily designed to be sand-tight if they are designed as current deflectors. But the quantitative benefits of groins acting as current deflectors are not well known.

Groins should only be used where it is known there is sufficient longshore sand transport. This implies that the littoral drift potential is high for the area, as well as the presence of a sufficient source of sand, in either the nearby eroding bluffs or on neighboring beaches. A good example of groins effectively holding the beach is at Case No. 23 in Anne Arundel County.

E. Alternate Approaches - Vegetative Control of Shore Erosion

On the basis of field visits and investigations by others, it appears that reliable vegetative means of erosion control are not available in areas where the erosive forces are substantial. This would include most portions of the main Chesapeake Bay shoreline which were described in this report. Although this study did not include an extensive analysis of vegetative types native to Chesapeake Bay, vigorous growth of marsh-type grass was found only at one location, the Hillsmere Beach site (Case No. 20), where it appeared that the vegetation could exert some mitigating effect on shoreline erosion.

Dean (1978) has reviewed the erosion-related physical effects of vegetation in the nearshore zone. These include the reduction and diversion of nearshore currents, thereby enhancing deposition. Phillips (1980) has attempted to develop planting guidelines for seagrasses and has noted the adverse effects of a high energy environment on seagrass survival. For example, tidal currents of 0.8 knots caused seagrass sprigs to be washed out in Great South Bay, New York. Plantings of "shoalgrass" appeared to be unable to resist the action of waves of 4 - 6 feet high at Port St. Joe, Florida. These wave heights are in the range that occur at least annually in Chesapeake Bay.

Wherever shore stabilization is considered a present necessity along the main body of the Upper Bay, the simple initiation and maintenance of a vigorous vegetative stand does not appear to be practical at present. The hydrodynamic forces acting on the Bay shoreline are increasing, particularly at most locations which already contain structures. This can be seen wherever flanking of an existing structure forms a protruding feature relative to the adjacent shoreline. This general increase in forces is due, in part, to sea level rise.

F. Alternative Approaches - Beach Nourishment

When sandy sediment is placed on a beach by either natural or artificial nourishment, the fine material will wash out until some equilibrium sediment size-distribution is reached under wave action. The amount of new material which must be placed on a beach to create one cubic yard of beach sand is referred to as the overfill ratio "K". Dean (1974) (Figure 4.1) shows calculations of overfill ratios for different size populations of native beach sand (n) and newly-emplaced material (b). The values for the x- and y- axis in Figure 4.1 are non-dimensional ratios of the mean grain-size to the standard deviations for the sediment populations on a beach, and for new beach fill.

For any program of beach nourishment along the Chesapeake Bay shoreline, an abundant source of suitable-sized material must be located, excavated, and transported to an erosion site. Substantial costs could be added to any beach nourishment project if sand has to be hauled more than a few miles to the shoreline; thus, sediments in the Coastal Plain which are located relatively close to the Bay would be the most likely terrestrial source for fill material.

To demonstrate the suitability of deposits which are exposed along the Bay shoreline for beach nourishment, four reaches were sampled to determine the size composition of the fastland sediments and of the beaches during the summertime. The four reaches were:

- bluffs in Calvert County between Cove Point and the BG&E Nuclear Power Plant,
- banks in Dorchester County on Taylors Island,
- banks in St. Mary's County between Point No Point and Cedar Point,
- bluffs in Kent County from Fairlee Creek to Swan Point.

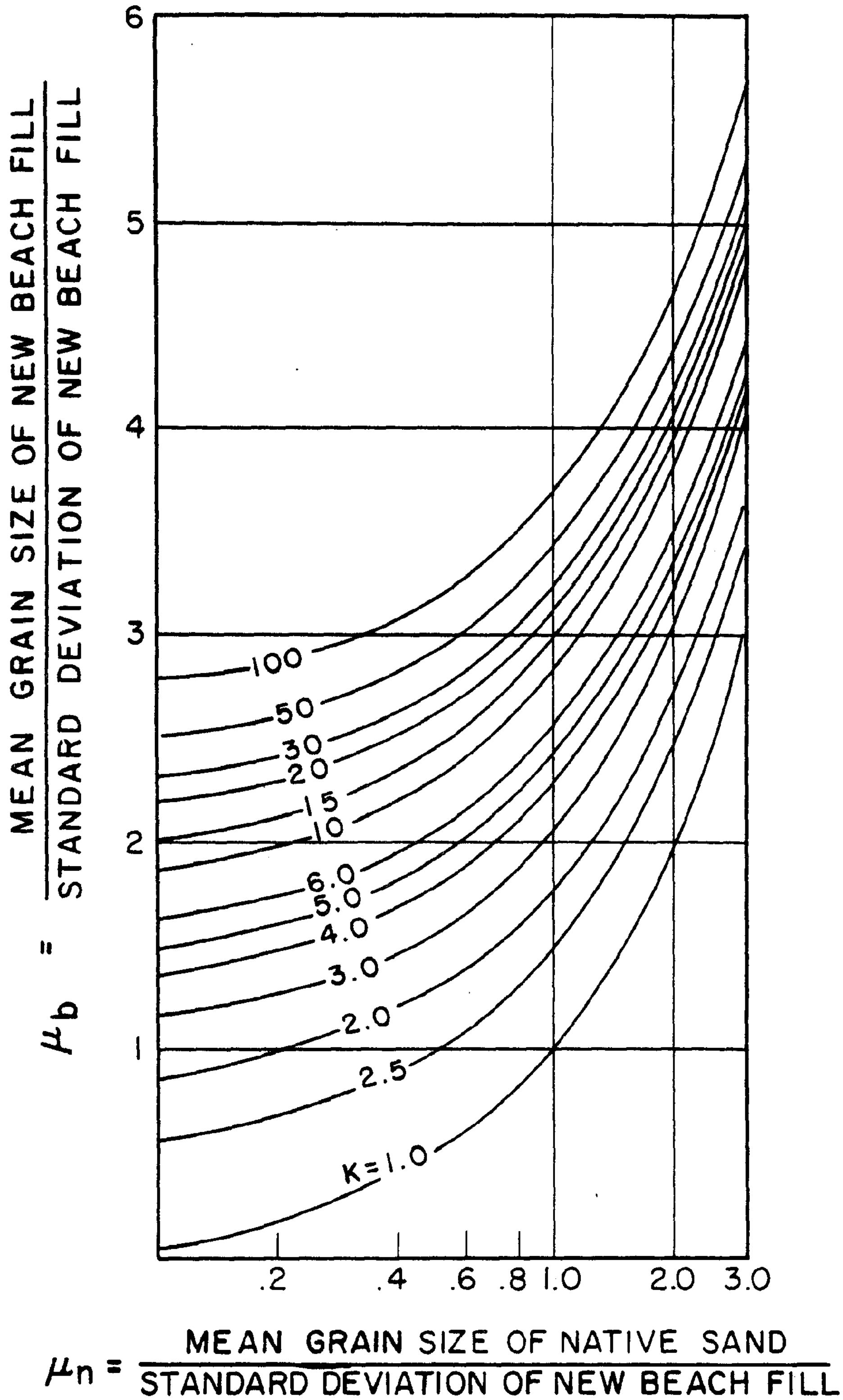
Representative shoreline profiles from these sites are shown in Figure 4.2.

Beach samples were collected in August 1980 from a trench dug across the beach and berm at each location. At the same time, materials from the adjacent bluff and banks were also sampled. Each bed of sand, or silt and clay, which could be discerned in outcrop was sampled at least ten times both vertically and horizontally. In the laboratory, the samples were wet-sieved to separate the sand and the weight of the sand was compared to the weight of silt plus clay. The sand fraction was then sieved again to further separate the size intervals of sediment.

Opposite: Figure 4.1. Plots of different overfill ratios "K" for different size populations of native beach sand and new beach fill.

Figure 4.1

PLOTS OF OVERFILL RATIOS "K"



Some of the results are shown in Figure 4.3, as plots of the mean grain size and the standard deviation (sorting) of the size distributions. The greatest size range in source sediment was found at the shoreline sites with the highest relief. More than one geologic formation (Appendix A) may be exposed in these shoreline areas, and this may contribute sediments in more size classes. This greater diversity of sediment size is particularly important for beach formation because the bluff sands are in most cases finer-grained than the beach sands along the same shoreline reach, and only a small percentage of the material eroded from the shoreline is sufficiently coarse to remain on the beach under summer wave energy conditions. The finer-grained sediments which do not remain on the beach are either transported offshore or alongshore to be deposited in spits or shoals in nearby coastal areas.

A determination of the amount of bluff material from these areas which actually remains on the beach can be computed from methods which are used to determine overfill ratios. The size statistics for the four sites are presented in Table 4.1, together with the appropriate value of "K" which can be read from the graph. The table also includes the historic rate of shore erosion at each site.

The results show the summer beaches at all four reaches were composed of medium to coarse-grained sands, and the material eroding from the adjacent shoreline is much finer-grained. The "overfill

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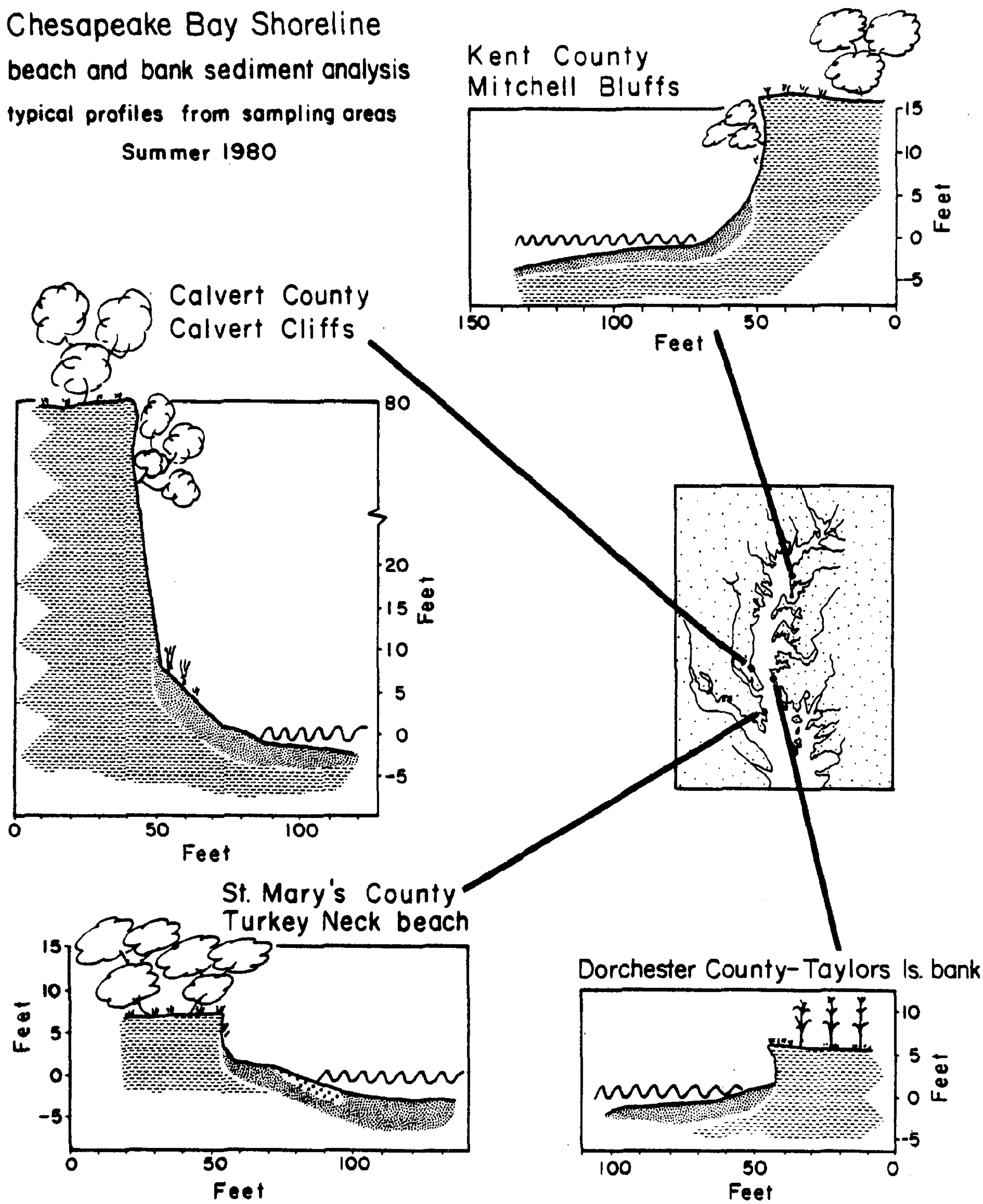
<u>Opposite:</u> Figure 4.2. Representative shoreline profiles in four areas which were sampled for sediment characteristics. The data from the samples is shown in Figure 4.3.

Figure 4.2



Chesapeake Bay Shoreline
beach and bank sediment analysis
typical profiles from sampling areas
Summer 1980
Kent County
Mitchell Bluffs
15
10
5
0
-5
Feet
150
100
50
0
Feet
Calvert County
Calvert Cliffs
80
20
15
10
5
0
-5
Feet
0
50
100
Feet
St. Mary's County
Turkey Neck beach
15
10
5
0
-5
Feet
0
50
100
Feet
Dorchester County-Taylors Is. bank
10
5
0
-5
Feet
100
50
0
Feet

ratio" of eroding shoreline sediments to volumes of native beach sand ranges between 5 and 10,000. Thus, at Taylors Island, only 0.01% of the eroded fastland sediments are remaining to form a beach, while 20% of the fastland sediments at Point No Point and Calvert Cliffs remain on the beach.

Table 4.1. Material Characteristics of the Four Test Reaches in Summer of 1980.

Reach	Bluff (b)			Beach (n)		K	Erosion Rate*
	μ	σ	% Silt	μ	σ		ft/yr
Pt. No Point	2	1.5	56	.9	1.1	5	2.9
Swan Point	2.2	1.0	78	.6	.9	27	3.3
Taylors Is.	3.0	.8	99	.8	.5	10,000	8.2
Calvert Cliffs	2.2	1.0	37	1.1	1.0	5	2.7

μ = mean grain size (ϕ unit)

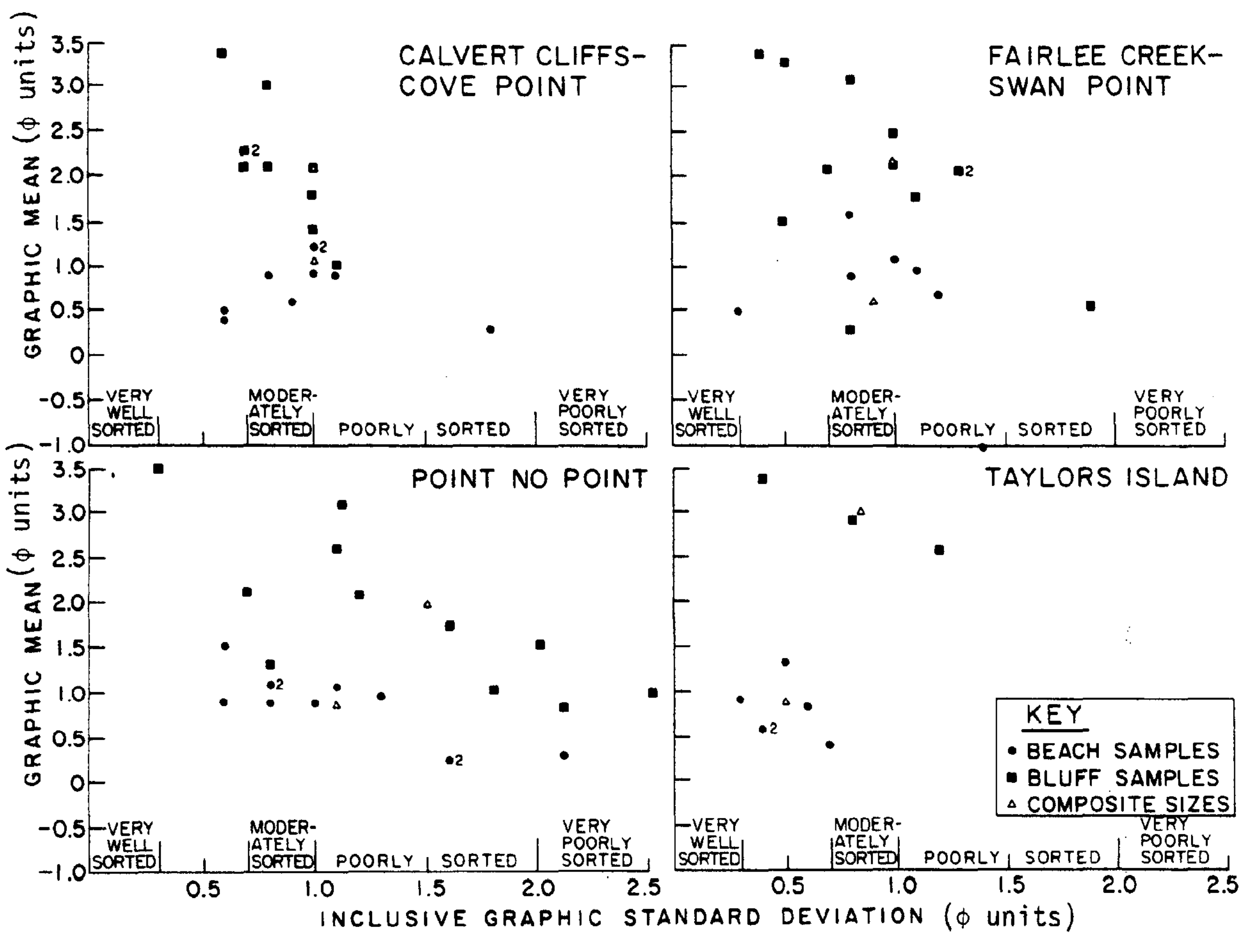
σ = standard deviation of grain size population (ϕ unit)

K = overfill ratio from Figure 4.1

* from Historical Shorelines and Erosion Rates (1975)

Opposite: Figure 4.3. Grain-size statistics of beach and bluff sediments from shoreline areas shown in Figure 4.2.

Figure 4.3



Grain Size Analyses of Shoreline Sediments from

Reaches Shown in Figure 4.2

The phi unit (ϕ) = $-\log_2 d$, where "d" is the grain diameter in millimeters.

In summary, the fastland material which was sampled in the four shoreline areas is much too fine-grained to remain on the beaches, indicating that the eroding shorelines in these test areas do not contribute much sand to develop protective beaches. There is a wide-variation in the actual overfill ratios from area to area; this result is due to small variations in the grain size statistics of the samples collected from outcrops right along the shoreline. More extensive sampling of the same strata in areas farther landward might disclose the presence of zones where coarser-grained sand is located. Finding the source of suitable sand which is available for excavation and transport to the beach at a reasonable cost is the first hurdle which must be surmounted in implementing a strategy of beach nourishment along the Chesapeake Bay shoreline.

Chapter V

RELATIONSHIP OF COASTAL PROCESSES TO HISTORIC EROSION RATES

Hsiang Wang, Robert Biggs,

Robert Dean, and Robert Dalrymple

A. Introduction

The information contained in this chapter and in Chapter VI describes the assessment of shore erosion on the northern Chesapeake Bay which was conducted as part of this study, along with the evaluation of erosion-control structures. Different sections of this chapter describe the coastal processes that are responsible for erosion, and the next chapter (Chapter VI) discusses a statistical analysis which examined all the factors for their relationship to the historic erosion rate. The purpose of the reach-by-reach description of coastal processes (contained in this chapter) and the statistical treatment (in Chapter VI) was to find those factors which could best explain the historic erosion rate, and thus produce a statistical model, or predictive equation, which planners could have used in assessing erosion on any portion of shoreline. Unfortunately, the results of the statistical analysis described in the next chapter indicate that modelling the pattern of historic erosion rates around the edges of the main Chesapeake Bay in Maryland cannot be suitably done by using traditional regression or discriminant analysis procedures.

The factors which were examined for their relationship to historic erosion rates include:

- shoreline terrain
- littoral drift of sediment

- tidal range
- storm surge
- rainfall
- wave climate

B. Historic Erosion Rates

Several years ago the Maryland Geological Survey (MGS) compared U.S. Coast and Geodetic Survey Charts dating back as far as 1841 with the latest available charts to show the linear recession of the Chesapeake Bay shoreline, and hence, the shoreline erosion rate. The original work was done by Singewald and Slaughter (1949). More recently, as part of Maryland's Coastal Zone Management Program, the MGS has updated and supplemented the earlier work with more map comparisons and field measurements from over 200 sites in Tidewater Maryland (from 1969-74). The results of this work are published in a map atlas entitled Historical Shorelines and Erosion Rates (1975). An example of the atlas product is shown in Figure 5.1. The entire atlas consists of all the 7 1/2 minute topographic quadrangle sheets for the Maryland portion of the Bay shoreline.

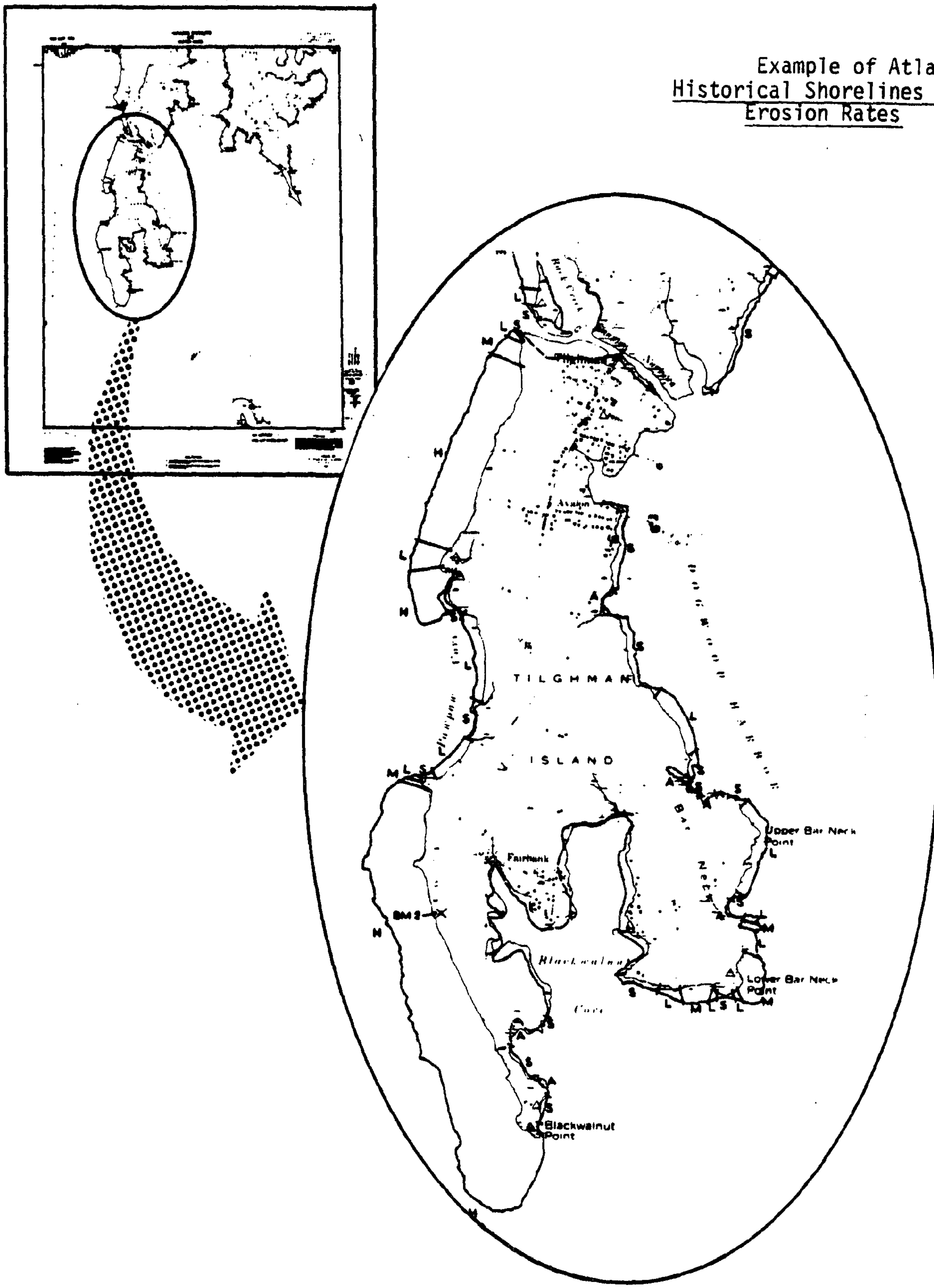
In these map reports, the erosion rate categories are designated in accordance with the following scheme:

Opposite: Figure 5.1. Example of atlas product showing historic shore erosion rates, derived from comparison of recent and historic charts maintained by U.S. Coast and Geodetic Survey, Rockville, Maryland (from MCZMP, 1975).

Figure 5.1

USGS 7.5" topo sheet

Example of Atlas
<u>Historical Shorelines and</u>
<u>Erosion Rates</u>



Slight	(S)	0 - 2 ft/yr.
Low	(L)	2 - 4 ft/yr.
Moderate	(M)	4 - 8 ft/yr.
High	(H)	> 8 ft/yr.
Fill	(F)	artificial fill
Accretion	(A)	

C. Highly-eroding reaches

Figure 5.2 shows the northern Bay shoreline broken down into reaches where the historic rate of erosion is generally "low" (< 4 feet/year), "medium" (4-8 feet/year) or "high" (> 8 feet/year) in the atlas Historical Shorelines and Erosion Rates (1975). It should be noted that the classification of historic erosion rate which is illustrated in Figure 5.2 represents only the gross characteristics within each reach. Considerable variations can exist within each reach; that is, if a reach is classified as highly erosional, it may contain sub- regions where the erosion is slight or even accretional. The classification is particularly subjective wherever the shoreline is highly irregular within the reach.

Nine reaches are identified as experiencing "high" erosion with average rates larger than 8 ft./year. These nine reaches are listed in Table 5.1. In addition, there are a few isolated points and small islands that are suffering severe erosion. There are also twenty-three reaches along the northern Bay margin that have experienced

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Opposite: Figure 5.2. Map showing generalized erosion rates in the northern Chesapeake Bay.

Figure 5.2

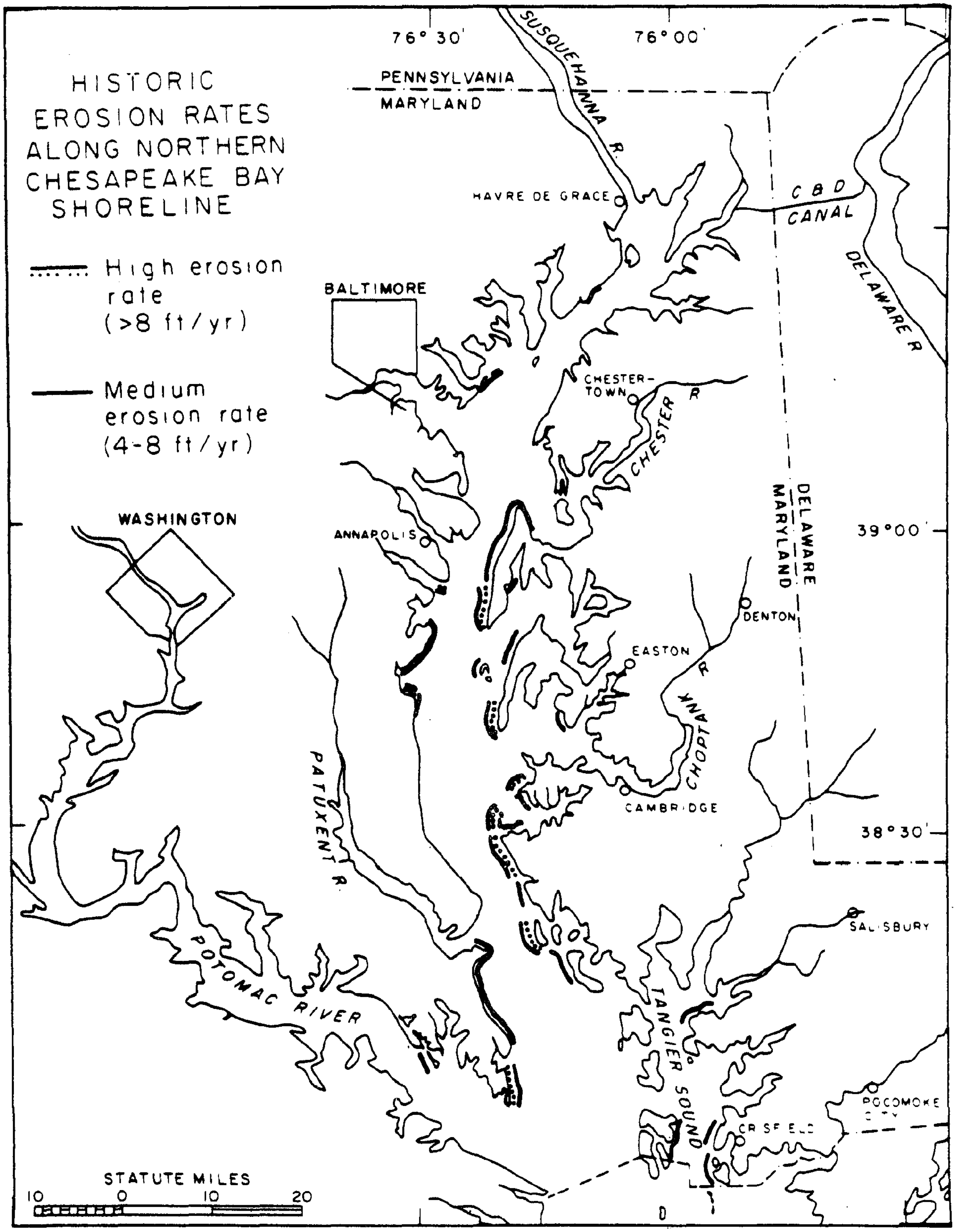

HISTORIC
EROSION RATES
ALONG NORTHERN
CHESAPEAKE BAY
SHORELINE
High erosion rate (>8 ft/yr)
Medium erosion rate (4-8 ft/yr)
76° 30'
76° 00'
39° 00'
38° 30'
PENNSYLVANIA
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C & D CANAL
DELAWARE R
BALTIMORE
CHESTER-TOWN
CHESTER R
DELAWARE
MARYLAND
WASHINGTON
ANNAPOLIS
DENTON
EASTON
CHOPTANK R
CAMBRIDGE
PATUXENT R.
SALISBURY
POTOMAC RIVER
TANGIER SOUND
POCOMOKE
STATUTE MILES
10
0
10
20

Table 5.1

Shoreline Reaches in Northern Chesapeake Bay

Experiencing High Erosion (> 8 ft/yr)

County	Location
St. Marys (Western Shore)	Pt. Lookout to St. Jerome Creek
Anne Arundel (Western Shore)	Holland Point
Anne Arundel (Western Shore)	Thomas Point
Queen Annes (Eastern Shore)	Kent Island - Craney Creek to Kent Point
Talbot (Eastern Shore)	Lowes Point to Knapps Narrows
Dorchester (Eastern Shore)	Mills Pt. to Hills Pt.
Dorchester (Eastern Shore)	James Island
Dorchester (Eastern Shore)	Oyster Cove to Punch Island Creek
Dorchester (Eastern Shore)	Barren Island

"medium" erosion ranging from 4-8 ft./year. Those reaches that are classified with either "high" or "medium" erosion total thirty- two reaches, or slightly more than a quarter of the entire shoreline in the northern Bay.

The following sections of this chapter explore the different ways in which one can evaluate the relationship between erosion parameters and the historic patterns of coastal retreat in the northern Bay.

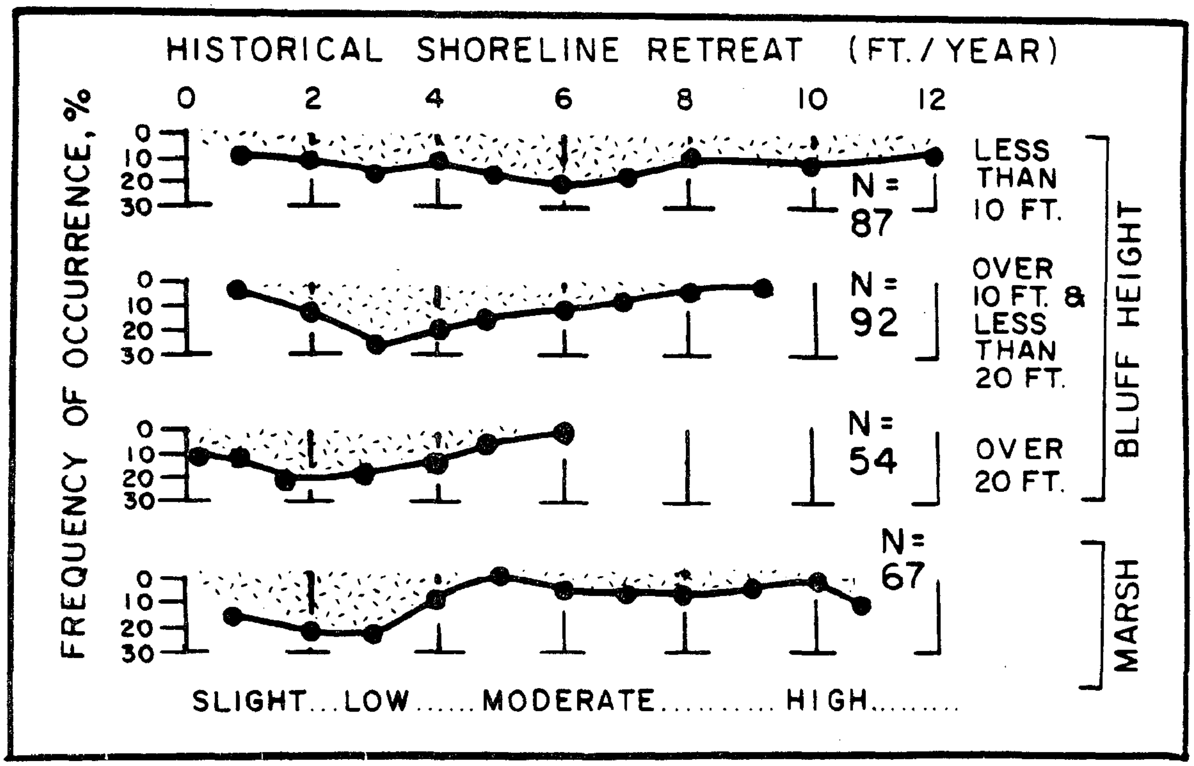
D. Relation of Shoreline Terrain and Geology to Coastal Retreat

The shore zone classification that follows was developed originally by Ahnert et al. (1974) for the Eastern Shore. For this study, the classification system was extended to the western shore of the Bay and its tributaries. The 1972 aerial photos (1:12,000) on file in the Wetlands Section of DNR were used to classify the shorelines on the western shore according to the terrain. Ahnert's (1974) data were used for the classification on the eastern shore. The complete classification is reported in a new atlas submitted to the Maryland Department of Natural Resources, which consists of transparent copies of all of the 7 1/2 minute topographic quadrangles of the Maryland portion of the Bay. An example of the new atlas product is shown in Figure 5.3. The categories used by Ahnert are defined as:

1. Shoreline without beach or bluff
2. Beach greater than 20 ft. in width
3. Beach against headland 0-20 ft. high
4. Beach against headland greater than 20 ft. high
5. Headland less than 20 ft. high, no beach
6. Headland greater than 20 ft. high, no beach
7. Fringe marsh (width between 0 and 100 ft.)
8. Intermediate width marsh (width between 100 and 400 ft)
9. Extensive marsh (width greater than 400 ft.)
10. Deltaic marsh (marsh containing mouth of tributary)

After the atlas product was completed, the shoreline terrain was compared to the historic erosion rates to show the relationship between these two factors. Figure 5.4 (below) is a graph which summarizes the historic erosion rates along all reaches of at least 0.5 kilometers in length which were also composed of only one type of shoreline terrain. The selection of 0.5 km. as a minimum length for study is arbitrary, but this is the smallest reach length which is regarded as suitable for analyzing variations in historical rates of coastal retreat.

Figure 5.4



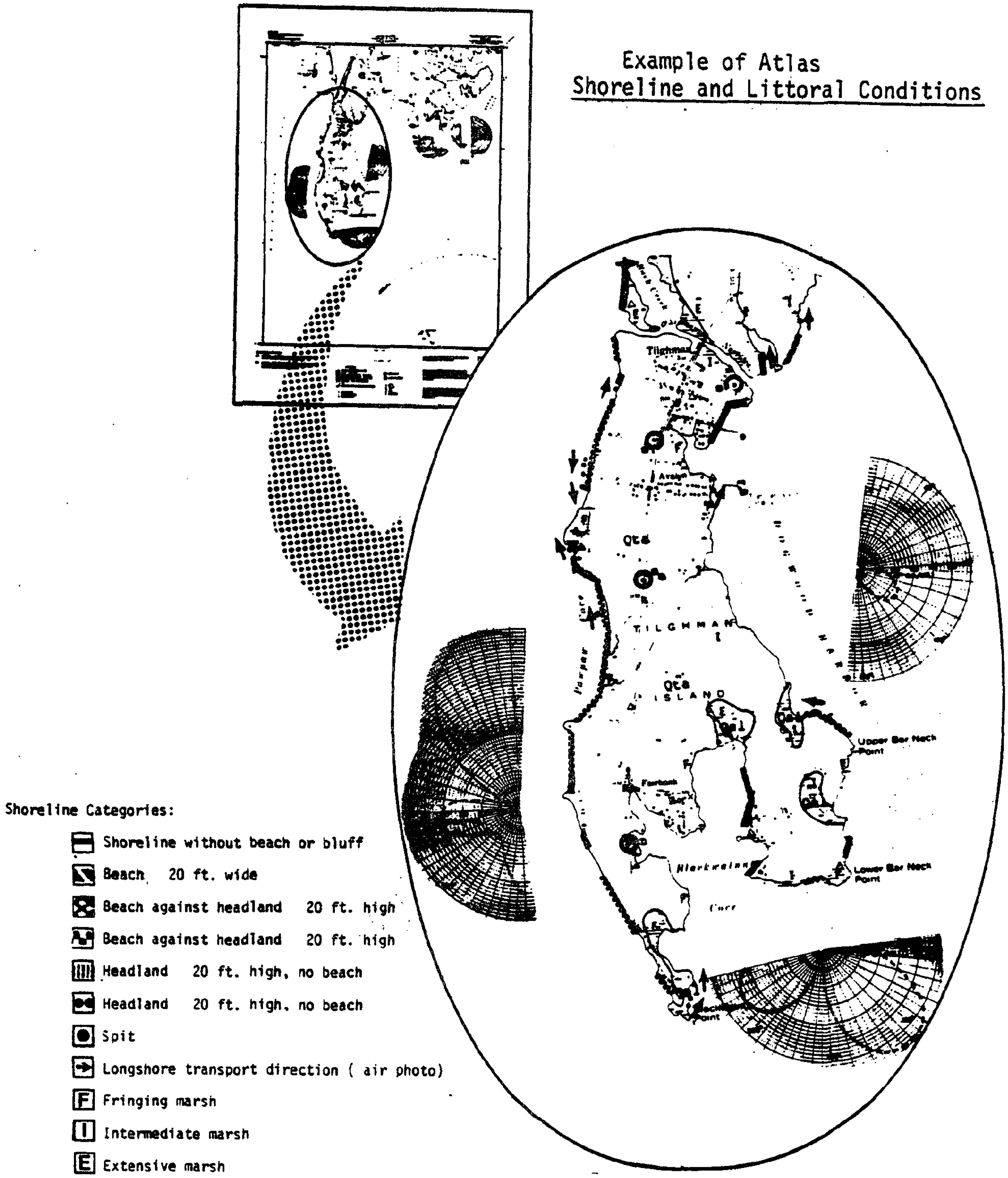
Above: Figure 5.4. Graph of relationship between the rate of coastal retreat and the shoreline terrain for the northern Chesapeake Bay.

Opposite: Figure 5.3. Example of atlas product showing shoreline classification, derived from aerial photographs.

Figure 5.3

USGS 7.5" topo sheet

Example of Atlas
Shoreline and Littoral Conditions



Littoral drift roses indicate potential longshore sediment movement from computer simulation not verified by field data.

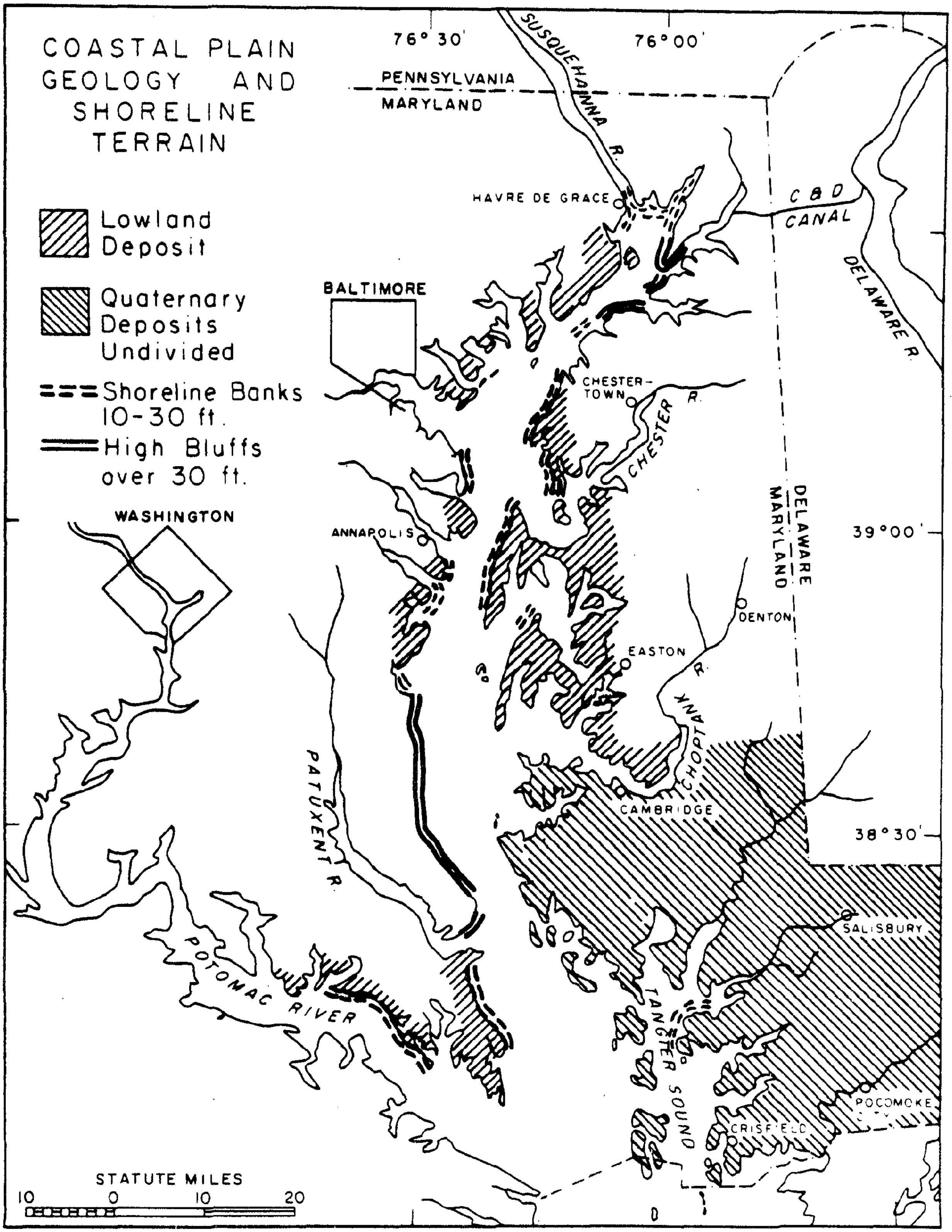
Figure 5.4 shows reaches possessing the highest historical rates of erosion are principally banks less than 10 feet high and some marshes. Most of the high bluffs are eroding at rates of 1-4 feet/year. Most of the marsh shoreline is also eroding at rates of 1-4 feet/year. Since there were a very small number of beaches at least 0.5 kilometers long on reaches where the historic rate of erosion was relatively uniform, these features were not included in the graph for comparison.

All the highly eroding reaches illustrated in Figure 5.2 can be compared with the distribution of bluffs and high banks which is shown in Figure 5.5, and with the broad categories of sediments which are exposed on different reaches. This figure shows all the highly-eroding reaches are located in regions of low relief, composed of either "Quaternary Lowland Deposits" or "Deposits Undivided" (see Appendix A). These materials are quite non-uniform and the textural characteristics of the exposures in some areas could be considered to have high resistance to erosion.

Unfortunately, not all of the reaches composed of the same geological formation are subject to "high" erosion rates. For instance, the bayside of Kent Island is highly erosional, yet the adjacent shoreline in Queen Annes County (north of the Chester River) has experienced only "low" shoreline retreat, although both regions have similar types of sediment exposed along the shore. The same relationship exists along many other shoreline reaches which have the same formations as those exposed in high-erosion areas, but possess low historic erosion rates instead.

<u>Opposite:</u> Figure 5.5. Map showing shoreline geology and shoreline terrain in the northern Chesapeake Bay.

Figure 5.5



COASTAL PLAIN GEOLOGY AND SHORELINE TERRAIN
Lowland Deposit
Quaternary Deposits Undivided
Shoreline Banks 10-30 ft.
High Bluffs over 30 ft.
76° 30'
76° 00'
39° 00'
38° 30'
PENNSYLVANIA
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C & D CANAL
DELAWARE R.
BALTIMORE
CHESTER-TOWN
CHESTER R.
DELAWARE
MARYLAND
WASHINGTON
ANNAPOLIS
DENTON
EASTON
CHOPTANK R.
CAMBRIDGE
PATUXENT R.
SALISBURY
POTOMAC RIVER
TANGIER SOUND
POCOMOKE
CRISFIELD
STATUTE MILES
10
0
10
20

E. Relation of Tide to Coastal Retreat

The astronomical tide in Chesapeake Bay is predominantly semi-diurnal with two high waters and two low waters per lunar day of 24.84 solar hours. Based upon tide records along Chesapeake Bay, Hicks (1964) constructed a mean range chart for the entire bay. Boon et al. (1978) have prepared a probabilistic model of the astronomical tide in Chesapeake Bay. Daily Mean Range and probability of occurrence of selected stations were computed for four classes shown in Table 5.2. Class 1 represents 12.5% of non-exceedance; Class 2 represents 37.5% of non-exceedance; Class 3 represents 62.5% of non-exceedance; and Class 4 represents 87.5% of non-exceedance. The Daily Mean Tide Range determined by Hicks is approximately equal to the average of values for Boon's Class 2 and Class 3 tides (see definition sketch on page 5-15).

For this study, the tidal range in the Bay between the stations listed in Table 5.2 was computed by a three-point interpolation procedure:

$$\eta(I) = \frac{1}{A_T}[\eta(1)A(1) + \eta(2)A(2) + \eta(3)A(3)] \qquad 5.1$$

where:

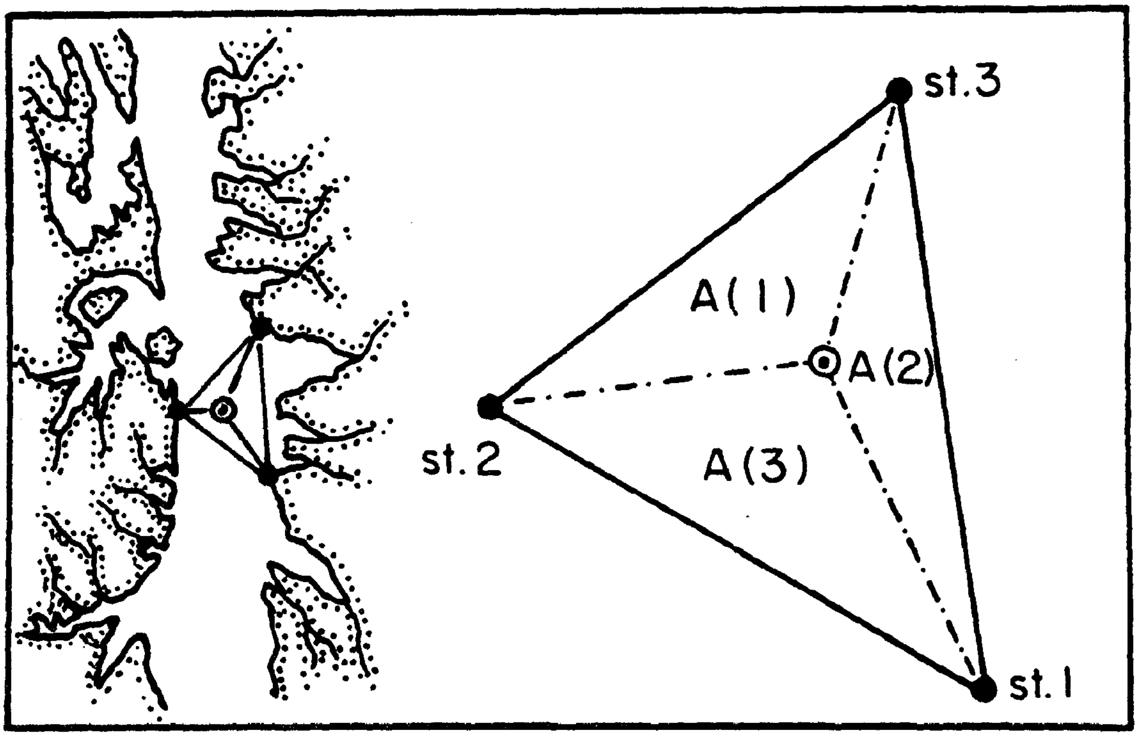
η is the tide range

$A = A(1) + A(2) + A(3)$, and

A is the area shown in Figure 5.6

Based upon this procedure, the tidal elevations inside the Bay were computed and the resulting mean tide range and Class 4 tide range (about 87.5% non-exceedance) were compiled onto a set of maps submitted to DNR. A simplified version of the maps are contained in Appendix B. The results show the tide range is quite moderate in the entire upper Bay. Below the Bay Bridge, the mean tide range is about 1.15 feet along the western shore and the Class 4 tide is about 0.25 feet higher. Along the lower Eastern Shore, the mean tide range increases to 1.5-1.6 feet. Above the Bay Bridge the differences in tide range between eastern and western shores diminish, and the tide range is generally less than in the lower Bay.

Figure 5.6



Above: Figure 5.6. Interpolation scheme for computing tide range solely from tide gauge data at known points.

Next Pages: Table 5.2. Class averages of Daily Mean Range and probability of exceedance values at selected stations in northern Chesapeake Bay (from Boon et al., 1978).

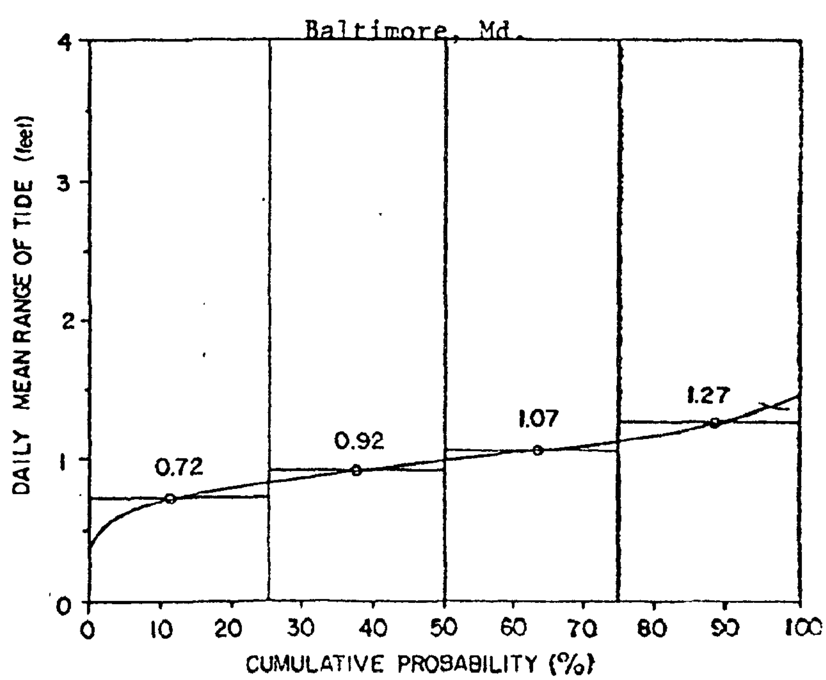
Table 5.2

Class Averages of Daily Mean Range (DMR) and Probability of Exceedance Values of Selected Stations in Chesapeake Bay From Boon *et al*. 1978)

Station	Class	DMR (feet)	Probability of Exceedance
1. Havre De Grace, MD	4	2.05	12.5%
	3	1.78	37.5%
	2	1.65	62.5%
	1	1.50	87.5%
2. Betterton, MD	4	1.78	12.5%
	3	1.59	37.5%
	2	1.47	62.5%
	1	1.30	87.5%
3. Tolchester, MD	4	1.37	12.5%
	3	1.19	37.5%
	2	1.06	62.5%
	1	0.86	87.5%
4. Baltimore, MD	4	1.27	12.5%
	3	1.07	37.5%
	2	0.92	62.5%
	1	0.72	87.5%
5. Love Point, MD	4	1.38	12.5%
	3	1.18	37.5%
	2	1.02	62.5%
	1	0.78	87.5%
6. Matapeake, MD	4	1.21	12.5%
	3	1.03	37.5%
	2	0.92	62.5%
	1	0.75	87.5%
7. Cambridge, MD	4	1.86	12.5%
	3	1.63	37.5%
	2	1.50	62.5%
	1	1.27	87.5%
8. Annapolis, MD	4	0.98	12.5%
	3	0.85	37.5%
	2	0.74	62.5%
	1	0.56	87.5%
9. Chesapeake Beach, MD	4	1.15	12.5%
	3	1.00	37.5%
	2	0.91	62.5%
	1	0.77	87.5%
10. Cove Point, MD	4	1.56	12.5%
	3	1.34	37.5%
	2	1.20	62.5%
	1	1.03	87.5%

Table 5.2 (Cont'd.)

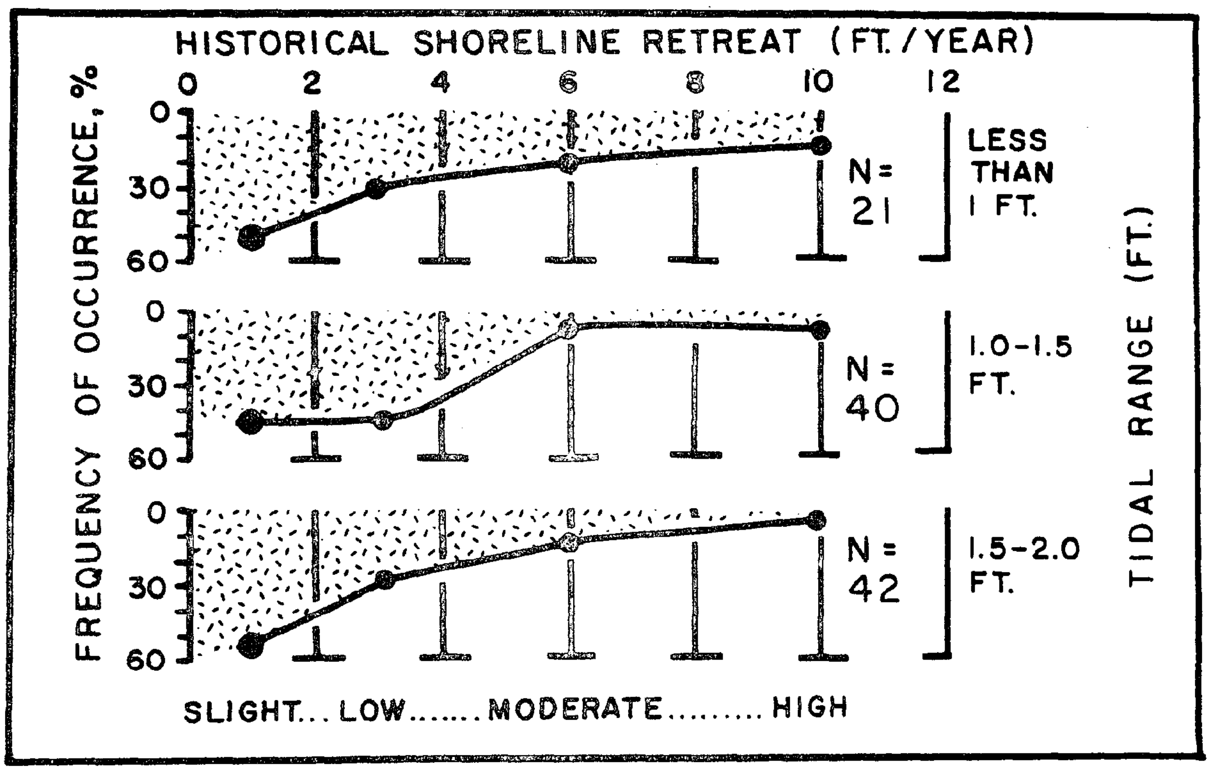
Station	Class	DMR (feet)	Probability of Exceedance
11. Solomons Island, MD	4	1.35	12.5%
	3	1.20	37.5%
	2	1.10	62.5%
	1	0.94	87.5%
12. Hoopers Island, MD	4	1.87	12.5%
	3	1.56	37.5%
	2	1.40	62.5%
	1	1.18	87.5%
13. Chance, MD	4	2.46	12.5%
	3	2.06	37.5%
	2	1.83	62.5%
	1	1.53	87.5%
14. Cornfield Harbor, MD (Point Lookout), MD	4	1.50	12.5%
	3	1.28	37.5%
	2	1.13	62.5%
	1	0.94	87.5%
15. Crisfield, MD	4	2.45	12.5%
	3	2.03	37.5%
	2	1.77	62.5%
	1	1.33	87.5%



Definition sketch for Baltimore

After computing the expected variations in tides throughout the upper Bay, the tidal range was compared to the historic erosion rate for all reaches at least 0.5 kilometers long which contained uniform erosion and tidal characteristics. The results are shown in Figure 5.7. Most of the reaches which were suitable for analysis possessed "low" rates of erosion. There are some slight differences in the curves shown in Figure 5.7, but there are no strong differences in the way tidal ranges are distributed between reaches with low, medium, or high historic erosion rates.

Figure 5.7



Above: Figure 5.7. Graph of relationship between the rate of coastal retreat and Class 4 tides for the northern Chesapeake Bay.

F. Relation of Storm Surges to Coastal Retreat

Based on their origin, major storms along the Chesapeake Bay can be classified into three major categories:

(1) Hurricanes and severe tropical storms,

(2) Extratropical cyclones or frontal wave disturbances over the mid-Atlantic and southeast coastal states,

(3) Wave developments along cold stationary fronts in the Gulf of Mexico west of $85^{o}W$ longitude.

Hurricanes and severe tropical storms are less frequent in the upper Chesapeake Bay but have the potential for producing higher surges because their great intensity can generate waves over the longest fetches. Extratropical cyclones, mostly occurring during winter periods, are more frequent but their intensities are usually far less. Since most extratropical cyclones have strong "northeaster" winds, the resulting storm surge in the upper Bay is less severe but more uniform over a wide area.

The frequency of occurrence and characteristics of different storms in the Chesapeake Bay region have been studied by many investigators. Brower et al. (1972) studied the distribution of tropical storms and hurricanes along the Atlantic coast from 1886-1957. During this period, a total of seventy-two tropical storms were observed in the Chesapeake Bay area. Their predominant direction was from the southwest and their occurrences were concentrated from June to October with August to October as the most vulnerable period. Boon et al. (1978) compiled a list of storms entering the Chesapeake Bay area between the years 1900 to 1977. A total of one hundred twenty-three tropical storms swept through this area over a record length of seventy-eight years. Therefore, based upon Brower's account, the Chesapeake Bay area experiences approximately one tropical storm per year whereas Boon's tabulation yielded a higher frequency on the order of 1.5 per year. The discrepancies arise from the following sources:

(1) Brower covered a period dating back to 1886 when the record was probably less than adequate.

(2) Boon counted many storms (a total of thirteen) twice because these storms changed directions during the period entering the area.

(3) Brower counted only observed storms, whereas Boon counted all the recorded storms.

In summary, the storm frequency for the upper Bay would be around 1 to 1.5 per year.

Chen (1978) has studied hurricanes and severe tropical storms for the effect of storm track on the storm surge. Three types of tracks are found to be of great interest in the Chesapeake Bay. One is a track passing to the left of the Bay center; another is the track passing to the right of the Bay center, and the third type is a track passing to the south of the Bay from east to west. These are denoted as HT1, HT2 (or HT2'), and HT3 respectively in Figure 5.8 .

Opposite: Figure 5.8 . Types of storm tracks in the Bay region.

Table 5.3. Some historical storm surges, compiled in the U.S. Army Corps *Chesapeake Bay Future Conditions Report*.

Figure 5.8

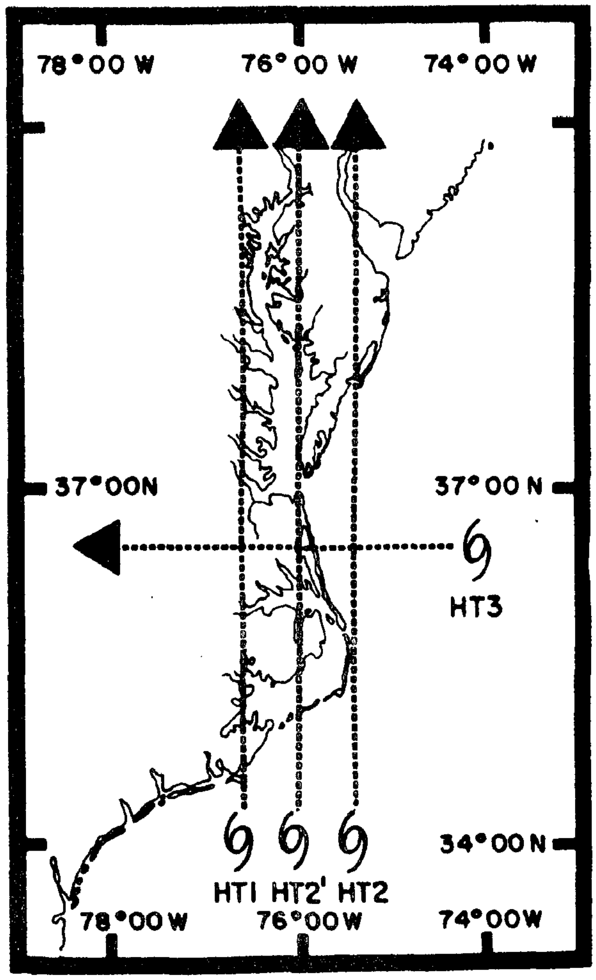


Table 5.3

RECENT CHESAPEAKE BAY STORMS

STORM	TIDAL ELEVATIONS (Feet Above Mean Sea Level) Norfolk	Mid Bay	Washington	Baltimore
August 1933	8.0	7.3	9.6	8.2
September 1936	7.5	—	3.0	2.3
October 1954 "Hazel"	3.3	4.8	7.3	6.0
August 1955 "Connie"	4.4	4.6	5.2	6.9
August 1955 "Diane"	4.4	4.5	5.6	5.0
April 1956 "Northeaster"	6.5	2.8	4.0	3.3
March 1962 "Northeaster"	7.4	6.0	—	4.7

In the upper Bay area, HT2-type storms occur rarely; thus, they are low frequency storms. But these types of storms are the most potent in creating high surges. The 1933 storm and the 1955 storm (Connie) which resulted in high storm surges in the upper Bay (Table 5.3) belong to this category. The difference in water heights from strong storm surges and those of extratropical cyclones (which are of higher frequency) can be seen by comparing the storm heights in Table 5.3 with the heights of surges compiled in Figure 3.2 (Chapter III) from the tide records at Baltimore, Annapolis, and Solomons Island.

Since the extratropical storms are far more frequent than the tropical storms, the surge data associated with extratropical storms are also expected to be far more statistically meaningful. Therefore, statistical interpretation of historical data can be used to produce meaningful information for storm surge with "annual" frequency, or with a frequency as high as 0.2 (with a return period of less than 5 years). On the other hand, the relatively small number of tropical storms that produced high surges of record at selected locations in the upper Bay is not sufficient to provide reliable statistical samples to predict low frequency storm surge levels. Therefore, a computer simulation model was used. This model was developed by Chen (1978) and has been applied previously to the prediction of surge in the lower Chesapeake Bay. The basic procedures to produce low frequency surge height curves by computer are as follows:

(1) Create synthesized wind fields by a five parameter hurricane wind model. The five parameters are: (a) central pressure, (b) radius of maximum wind, (c) forward speed, (d) angle of

approach, and (e) track. Each of these five parameters has an associated probability of occurrence which is combined to estimate the frequency of storms.

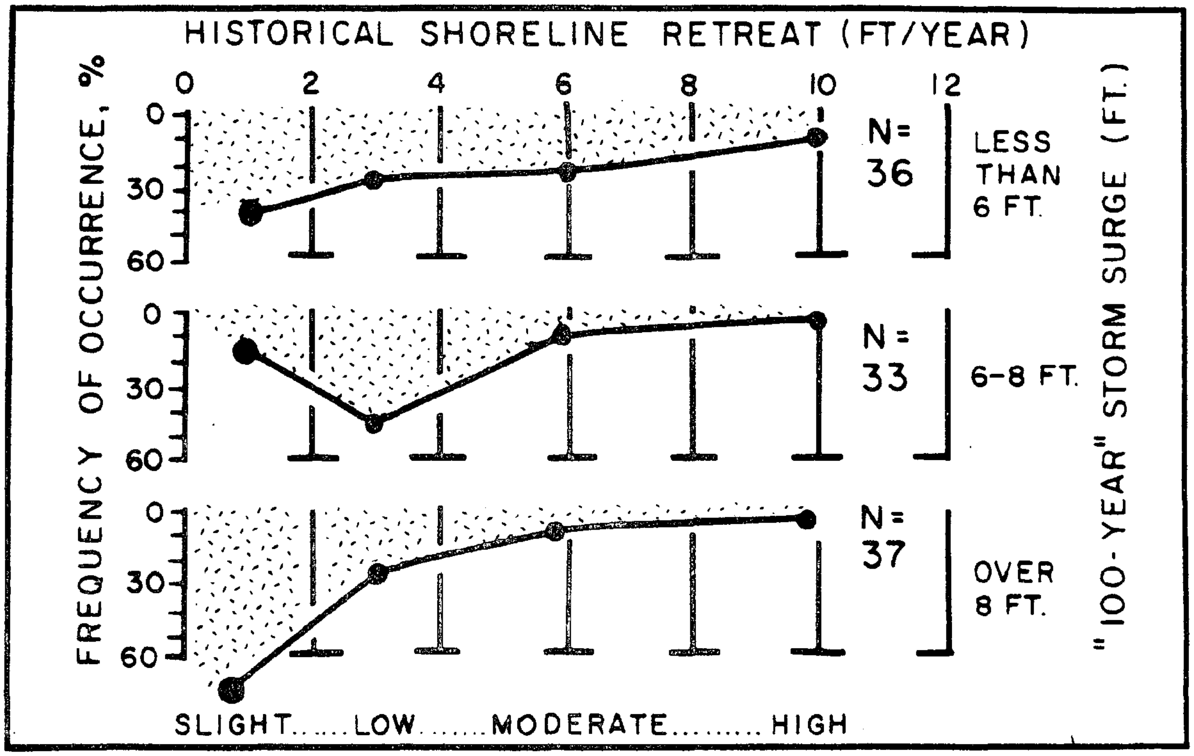
(2) The synthesized wind field is next used to drive a finite element hydrodynamic model to compute the storm surge. The storm surge computations are repeated for storms with various parametric combinations but with the same joint probability of occurrence.

(3) The maximum storm surge levels at selected locations are combined with a probabilistic astronomical tide model to produce the final computer projections of storm surge heights in the upper Bay.

The results for a "100-year" storm surge are plotted in Figure 5.10. Predicted "100-year" storm surge heights were also compiled onto a set of atlas maps submitted to the Maryland Department of Natural Resources. A simplified version of the map is contained in Appendix B. As expected, the surge level increases progressively towards the head of the Chesapeake Bay. The predicted surge level is about 4.6 feet near the mouth of the Potomac River and increase to over 11 feet at Havre de Grace.

After deriving these computer projections for the "100-year" storm surge, the surge levels were compare for all reaches at least 0.5 km. long where both the historic erosion rate and storm surge levels were uniform. The results are plotted in Figure 5.9 . As in the case of the comparison based on tides (Figure 5.7), most of the reaches suitable for analysis possess "low" historic rates of coastal retreat, and there are no general differences in the way predicted "100-year" storm surge levels are distributed between reaches with low, medium, or high historic erosion rates.

Figure 5.9



<u>Above:</u> Figure 5.9. Graph of relationship between the rate of coastal retreat and the predicted "100-year" storm surge height for northern Chesapeake Bay.

<u>Opposite:</u> Figure 5.10. Height-Frequency estimates of storm surges for the Maryland Bay shore.

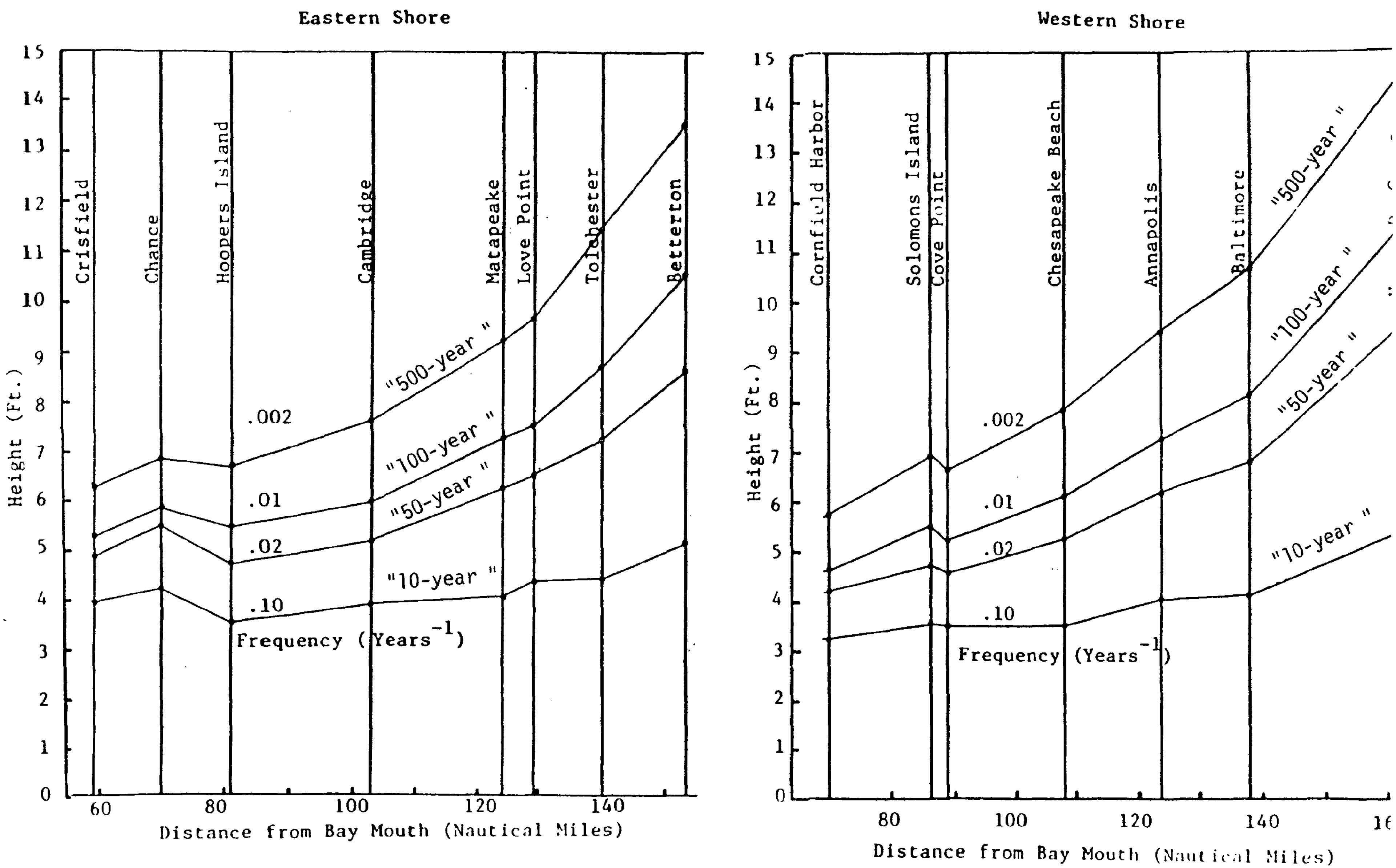


Figure 5.10 Height-Frequency Estimates of Storm Surge, Maryland Bay Shore.

G. Relation of Wave Climate to Coastal Retreat

Wave conditions near the shoreline and the directions of wave energy flux are probably the most important factors which are needed to assess erosion potential. In northern Chesapeake Bay, only limited ship-observed wave data are available. They are visually-observed data reported by transiting ships. These data are inadequate for use in categorizing the wave climate on shorelines for a number of reasons:

(1) The data are insufficient to be statistically meaningful.

(2) Most of the observations are from the vicinity of main shipping channels whereas the main interests in applying the wave observations is along the shorelines.

(3) Sporadic visual observations of wave heights tend to be biased to the median range waves; both high and low waves are often less than adequately recorded.

Therefore, to establish wave statistics for erosion assessment a wave hindcast numerical model was used. This model is based on a shallow water wind-wave generation technique developed by Wilson (1965). For restricted and shallow fetches, it is necessary to compute wave heights using a computer model in order to include all important effects, such as bottom friction, irregular fetch areas, and wave breaking. The computer model developed by COER, Inc. is based on an empirical wave height and period formula of Wilson (1965) and a procedure outlined by St. Denis (1969).

The basic equations are:

$$\frac{gH}{U^2} = 0.30\left[1 - \left\{1 + 0.004\left[\frac{gF}{U^2}\right]^{1/2}\right\}^{-2}\right] \qquad 5.1$$

and

$$\frac{gT}{U} = 8.60\left[1 - \left\{1 + 0.008\left[\frac{gF}{U^2}\right]^{1/3}\right\}^{-5}\right] \qquad 5.2$$

Here "H" is the significant wave height (defined as the average height of the highest third of the waves) and "T" is the corresponding wave period; "g" and "U" are gravity and wind speed, respectively, and "F" is the fetch (the distance over which the wind blows). These equations are based on North Sea wave data and are valid for an infinite depth.

To incorporate depth effects, a reduction factor "r" is introduced to reduce the wave height as a result of bottom friction.

$$r = \left[1 + \frac{\phi f H}{T^4}\,\Delta x\right]^{-1} \quad \text{where} \quad \phi = \frac{64\pi^3}{3g^2}\,\frac{K}{\sinh kh} \qquad 5.3$$

and

$$K = \left[\tanh kh\left[1 + \frac{2kh}{\sinh 2kh}\right]\right] \qquad 5.4$$

where "k" = wave number (2 /L) and "h" is the water depth. The factor "f" is a friction factor, which generally is taken as 0.01 for sandy areas. The x is the distance travelled by the wave over a shallow bottom of depth "h". For computational purposes, a given distance is broken down into numerous lengths "Δ x" of assumed constant depth.

The computational procedure consists of a number of steps.

1. The water surface is overlain by a fan of rays emanating from the shoreline point of interest. This fan is symmetrically

distributed above the wind direction. Along each ray, the depth is determined at a number of points distributed at a distance "Δx".

2. For a particular section along a ray, the wave height entering the section is taken as known. (It either is zero, at the upwind beginning of the ray, or it is known from the previous upwind section). The equivalent fetch length is determined for that wave height, by solving Equation 5.1 for "F". For the new section, a new "F" is computed by adding "Δx". A deep water wave height and period are then obtained for the section using the two equations.

3. The wave height reduction factor is then calculated and used to reduce the wave height "$H = rH_d$".

4. If the wave height exceeds the breaking wave height, the wave height is reduced to "H = h" where " " = 0.8 and "h" is the average depth over the section.

5. Move downwind to the next section and compute wave height and period there, etc., until the shoreline point is reached. Repeat for all rays.

6. The significant wave height and period generated at a point is composed of the contribution from all the rays. Therefore

$$H_i = \frac{\sqrt{\sum_{i=1}^{N} H_i^2 \cos^2\alpha_i}}{\sqrt{\sum \cos^2\alpha_i}} \; ; \qquad T_s = \frac{\sum_i T_i (H_i \cos\alpha_i)^2}{\sum_i H_i^2 \cos^2\alpha} \qquad 5.5$$

where "α_i" is the angle the ray "i" makes with the wind direction, and "H_i" is the wave height at the shoreward most section.

"Annual" Wave Statistics - The distributions of "annual" wave height and direction were computed for the complete shoreline of the northern Chesapeake Bay. The wind input information was based on the long-term statistics at three stations--Baltimore, Annapolis, and Patuxent River which represent artificial divisions of the upper Bay into three regions: in the upper northern-most region the Baltimore Airport wind data were used; in the middle region the Annapolis data were used; and in the lower northern region the computations were based on the Patuxent River wind information.

The complete "annual" wave statistics are presented graphically in atlas form as wave roses that plot the expected significant wave height versus direction. These atlas maps have been supplied to the Maryland Department of Natural Resources. The "annual" wave climate is composed principally of waves whose heights are on the order of 0.5 - 1 foot, and should be regarded with less importance than storm-wave conditions in assessing erosion and the performance of shoreline structures.

Storm Wave Conditions - One of the important tasks in the present study which is discussed at some length here is the derivation of the storm wave conditions. The heights of these waves were plotted in the case studies in Chapter II at the sites of various kinds of structures on the Chesapeake Bay shoreline. Two kinds of storm-wave conditions were studied: the storm waves due to tropical hurricanes, and the storm waves due to "northeasters". In the computations, the same wind-wave computer program was used for the annual wave climate, except that the input wind and initial hydrographic conditions are different.

Storm Waves Due to Extratropical Storms - As stated earlier, the extratropical storms are associated with low return periods of 10 years or less. For this type of storm waves, the following input conditions are used:

Wind Speed : Uniformly over the water.

Wind Direction : North, northeast, and east for the western shore and north, northwest and west for the eastern shore.

Surge Elevation: Storm surge as obtained from historic tide records illustrated in Figure 3.2 added to MSL.

Storm Waves Due to Tropical Storms - For the case of low-frequency storm waves (here defined as storms of return period higher than 10 years), the input conditions are more difficult to define. This is because the hurricane wind model is not unique but is defined by five parameters. In theory, by various combinations of these parameters, one can create an infinite number (or at least a large number) of synthesized wind conditions that are compatible to the "100-year" storm in a statistical sense. For a designated location, one of these storms will produce the most severe wave conditions. Thus, to determine the extreme wave condition for a "100-year" storm, one should test all the possible cases and then identify the most severe one among them. Such an approach is, of course, impractical. For the present study, only one synthesized storm was used. This synthesized "100-year" storm is selected to be compatible to the storms that have produced the highest surges in the upper bay. Based upon the historical records, large storm surges in the Chesapeake Bay were usually produced by slow-moving landfall storms of type HT2 (Figure 5.8) with large wind radius. These types of storms which generate high surges are assumed to generate high waves in the same region also.

Based upon the above observations, the synthesized "100-year" storm which was selected for the wave computations assumes the following basic characteristics:

- Wind direction - South, southwest or southeast.
- Wind radius "R" - 40 nautical miles per hour. This is the mean value of a large radius storm compatible to historical storms causing high surges in the upper bay.
- Forward speed "V_F" - 12 knots, or stalling.
- Wind strength -

  Maximum wind speed "V_p" - 90 knots (104 mph) for 100-year storm.

  Maximum hourly wind speed (V_m) - 78 knots (90 mph)
- Wind field - assumed to be an idealized Rankine vortex expressed as:

  $$V = \frac{V_m}{R} r \quad \text{for} \quad r < R$$ (rotational core), and

  and

  $$V = \frac{V_m R}{r} \quad \text{for} \quad r \geq R$$ (irrotational outer region)

  where "R" is the radial distance from the hurricane center. See Figure 5.11.

The wind strength in the above synthesized storm is determined in accordance with the procedure recommended by the U.S. National Weather Service (1972). The maximum wind "V_p" is obtained by

$$V_p = 0.868\ K\ (P_n - P_o)^{1/2} - 0.5\ Rf \qquad 5.6$$

where:

V_p is the maximum wind speed in knots;

P_n and P_o are peripheral pressure and central pressure, respectively, in inches of mercury (H_g);

R is the radius to maximum wind in nautical miles;

f is the Coriolis parameter = 0.525 sin/hour with the latitude; and

K is a constant approximately equal to 73.

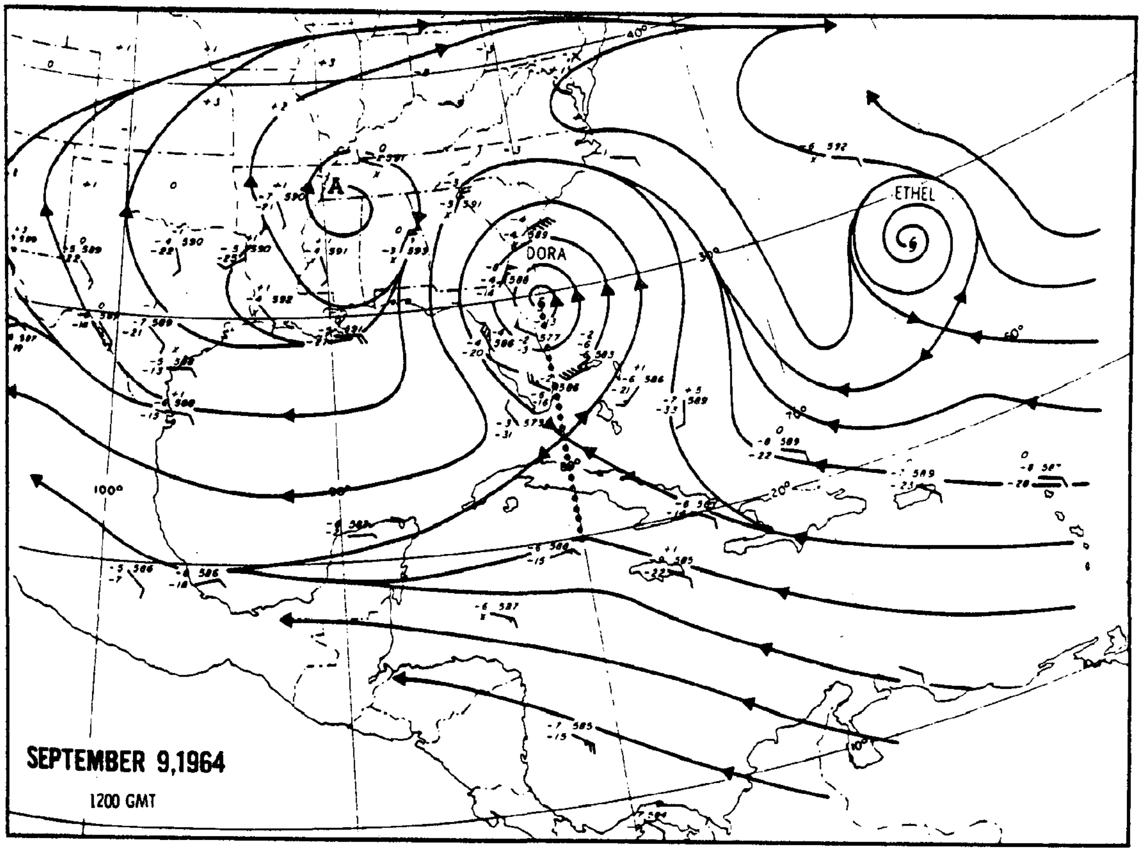
Based upon the design graphs prepared by the U.S. National Weather Service, "P_o" should be around 27.6 inches Hg for a "100-year" storm in the Chesapeake Bay region. The corresponding "V_p" should then be on the order of 90 knots. The maximum hourly wind "V_m" can be estimated as:

$$V_m = 0.865\ V_p \qquad 5.7$$
$$= 78 \text{ knots or } 90 \text{ mph}$$

This value corresponds reasonably well with the value for extreme conditions extrapolated from the wind records from Baltimore Washington International Airport.

Opposite: Figure 5.11. Wind field in idealized hurricane as a function of radius. Also shown is an example weather map for a hurricane.

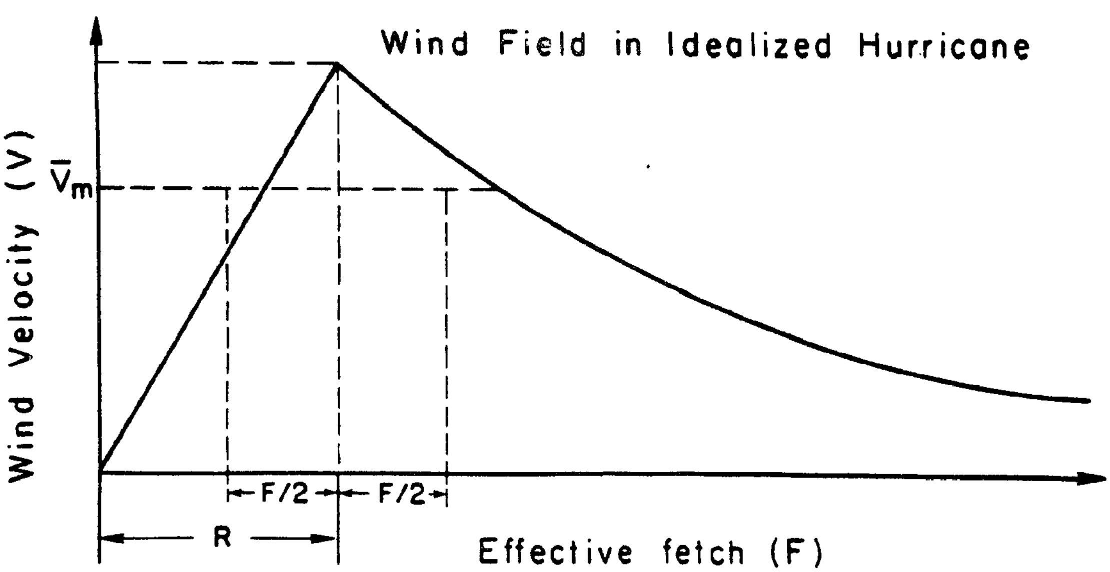
Figure 5.11



Hurricane "Dora" Streamlines

Figure 5.11

Wind Field in Idealized Hurricane



With this hypothetical wind field, the storm waves at any location are generated by the following procedures:

(1) The appropriate storm surge height is added to the astronomical tide plus MSL to determine the water depths under storm conditions.

(2) Depending upon the location, the wind direction is selected from the three possible choices such that it coincides with the longest effective fetch. For instance, if a certain reach has effective fetches from the south, southeast and southwest of 25, 32 and 29 nautical miles respectively, then a wind direction from the southeast is selected.

(3) Since the wind-wave generation model assumes a homogeneous wind field, the mean wind strength over the effective fetch is estimated by the following equation:

$$\bar{V}_m = \frac{V_m}{F}\left[\int_{R-\frac{F}{2}}^{R} \frac{r}{R}\,dr + \int_{R}^{R+\frac{F}{2}} \frac{R}{r}\,dr\right] = \frac{V_m}{2}\left\{\left(1 - \frac{F}{4R}\right) + \frac{2R}{F}\,\ell n\left(1 + \frac{F}{2R}\right)\right\} \qquad 5.8$$

The physical meaning of "V_m" is shown in Figure 5.11. It represents the mean hourly velocity over a region that spans one-half the effective fetch length from either side of the peak velocity location.

(4) Since the strength of the wind over the generation area will gradually diminish due to either the storm moving out of the region or energy dissipation, one must check whether the wind-wave generation process is duration limited. To examine this possibility, the effective wind duration "t_e" during which the wind velocity maintains "V_m", must be estimated. For a moving storm with forward speed "U_F", the value for "t_e" is obtained simply as:

$$t_e = F/U_R \qquad 5.9$$

For a stalled storm, a storm histogram must be known or assumed. For the present study "U_R" is taken as 12 knots as mentioned earlier. This effective wind duration is now compared with the minimum wind duration "t_{min}" required to generate the fully risen sea. If "t_e" is greater than "t_{min}", the generation is duration-limited.

(5) The above input information was fed into the computer model to obtain the storm wave conditions. The resulting storm wave conditions were used with surge predictions to compute the levels of "Run-Up" for the 40 case studies in Chapter II.

The results of the storm wave computer forecasts are presented graphically together with the wave forecasts for "annual" storms in atlas form as wave roses that plot the expected significant wave height under storm wind conditions versus direction. These atlas maps have been supplied to the Maryland Department of Natural Resources. Some idea of the kind of wave energy distribution which is predicted around the upper Bay is illustrated in the map in figure 5.12. The wave energy is arbitrarily categorized as "medium" if the maximum wave height during an "annual" storm is between 2.5 and 4.0 feet. The wave energy is considered "high" if the maximum wave height during an "annual" storm is over 4.0 feet.

All the highly-eroding reaches illustrated in Figure 5.2 can be compared with the distribution of wave energy in Figure 5.12. This comparison shows most of the highly-eroding zones are situated in zones of "medium" to "high" wave energy. But the reverse is not true. The shoreline along Calvert County, for instance, is in "high" wave energy, but the historic rate of coastal retreat is generally low.

After deriving the computer forecasts of wave climates, the general variations in wave energy were compared to the historic erosion rate for all reaches at least 0.5 km. long which contained uniform erosion and wave characteristics. The results are shown in Figure 5.13.

As in the comparisons of storm surge and tide range, most of the reaches which again were of sufficient length to be included in the analysis have "low" historic rates of coastal retreat. Like the

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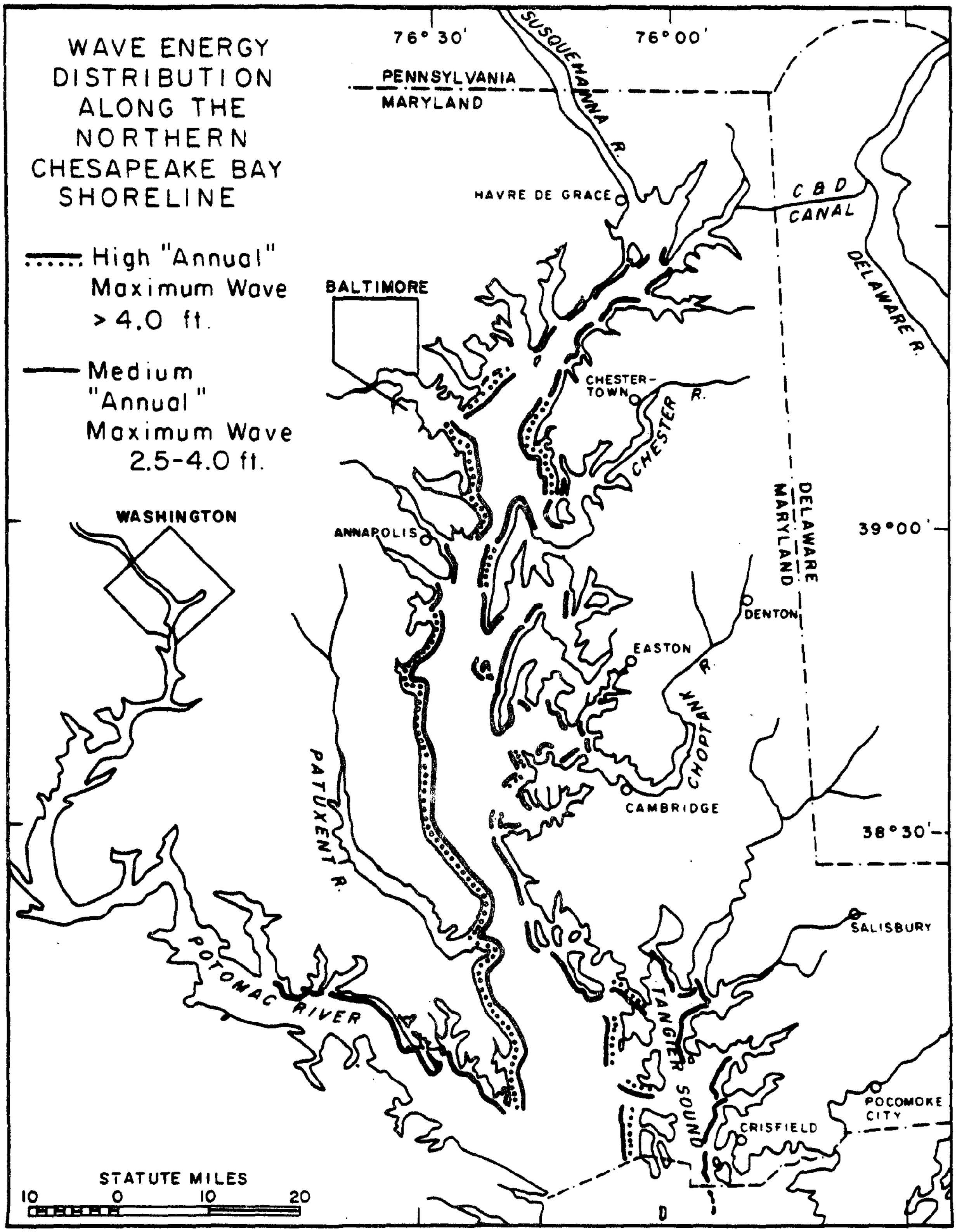
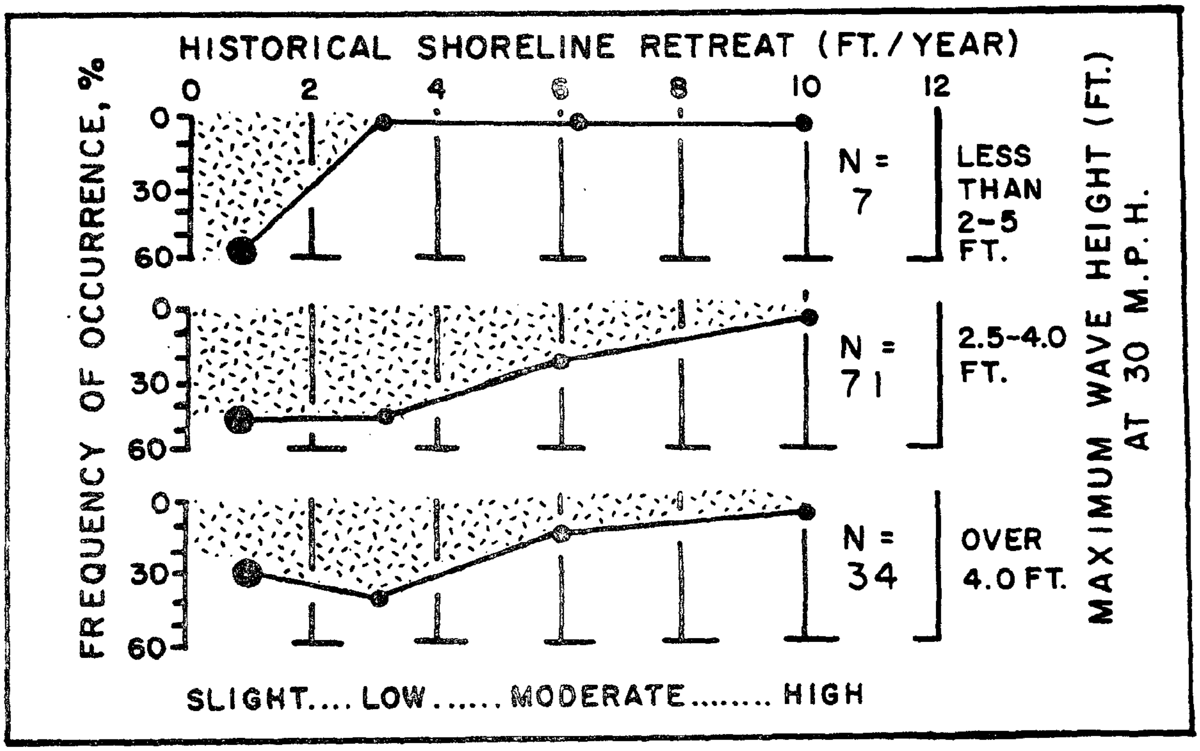
<u>Opposite:</u> Figure 5.12. Map showing distribution in wave energy in the Northern Chesapeake Bay.

Figure 5.12


WAVE ENERGY DISTRIBUTION ALONG THE NORTHERN CHESAPEAKE BAY SHORELINE
High "Annual" Maximum Wave > 4.0 ft.
Medium "Annual" Maximum Wave 2.5-4.0 ft.
76° 30'
76° 00'
PENNSYLVANIA
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C & D CANAL
DELAWARE R.
BALTIMORE
CHESTER-TOWN
CHESTER R.
DELAWARE
MARYLAND
39°00'
WASHINGTON
ANNAPOLIS
DENTON
EASTON
CHOPTANK R.
CAMBRIDGE
PATUXENT R.
38°30'
SALISBURY
POTOMAC RIVER
TANGIER SOUND
POCOMOKE CITY
CRISFIELD
STATUTE MILES
10
0
10
20

graphs in Figures 5.7 and 5.9 , the graph in Figure 5.13 reveals no general differences in the way wave climate is distributed between reaches with low, medium, or high historic erosion rates. The pattern emerging from this approach to evaluating erosion suggests that none of the factors necessary to explain erosion (waves, tides, storm surges) are by themselves sufficient to explain why any reach has had a "high" or "low" historic erosion rate.

Figure 5.13



<u>Above:</u> Figure 5.13. Graph of relationship between the rate of coastal retreat and the distribution of wave heights on the northern Chesapeake Bay.

H. Relation of Littoral Drift to Coastal Retreat

Longshore sediment transport is the movement of sand more or less parallel to the shoreline due to waves approaching the shoreline at an angle. The dominant effects of the waves breaking are to: (1) mobilize the bottom sediment, and (2) cause a weak longshore current in the "downcoast" direction, see Figure 5.14. In the Bay where longshore currents may be due to other causes, primarily tides, these currents also cause sediment transport. The direction of the longshore sediment transport changes with time depending on the winds (which generate the waves) and possibly on tidal currents. In some places, there may be a seasonal variation in the longshore sediment transport, and the transport direction may be highly irregular, depending on individual storms. If structures partially or completely impede the longshore sediment transport, the sand will deposit on the up-drift side of the structure, thereby leaving a signature of the direction of recent longshore sediment transport.

It is useful to define an unambiguous direction for longshore sediment transport. For purposes here, the definition used in the Shore Protection Manual (1973) will be adopted; that is, the longshore sediment transport is positive if it moves to the right of a shore-based observer. Thus, in Chesapeake Bay on the eastern shore, "positive" transport is to the north and on the western shore, to the south. This is a purely arbitrary notation.

The capacity of the waves to cause longshore sediment transport, Q_s, is generally expressed as

$$Q_s = K H_b^{5/2} \sin 2 \alpha_b \qquad 5.10$$

in which

Q_s = capacity for longshore sediment transport in cubic yards per year <u>if</u> there is adequate sediment to be transported,

K is an empirical constant, $K = 3.5 \times 10^5$,

H_b is the breaking wave height in feet, and

α_b is the breaking wave direction relative to a beach normal (see Figure 5.14).

If only the longshore sediment transport is of importance, it is possible to relate changes in longshore sediment transport "Q_s" to volumetric erosion rates. For example, referring to Figure 5.15, if the sediment transport rate "Q_B" out of a region of interest is greater than the sediment transport rate "Q_A" into the region of interest, volumetric erosion "V" will occur at a rate

$$\frac{\Delta \forall}{\Delta t} = Q_B - Q_A \qquad 5.11$$

There are several measures of longshore sediment transport that are of interest. The <u>net</u> longshore sediment transport "Q_N" is either "positive" or "negative" and is given by

$$Q_N = Q_+ - Q_- \qquad 5.12$$

where "Q_+" and "Q_-" are the magnitude of the "positive" and "negative" longshore transport rates, respectively. If a groin represents a complete littoral barrier installed on a uniform beach, the

---

<u>Opposite:</u> <u>Above:</u> Figure 5.14. Longshore current resulting from waves breaking at an angle to shoreline.

<u>Below:</u> Figure 5.15. Illustration of continuity equation 5.11 (after Dean, 1976).

Figure 5.14

SCHEMATIC DIAGRAMS OF LITTORAL DRIFT

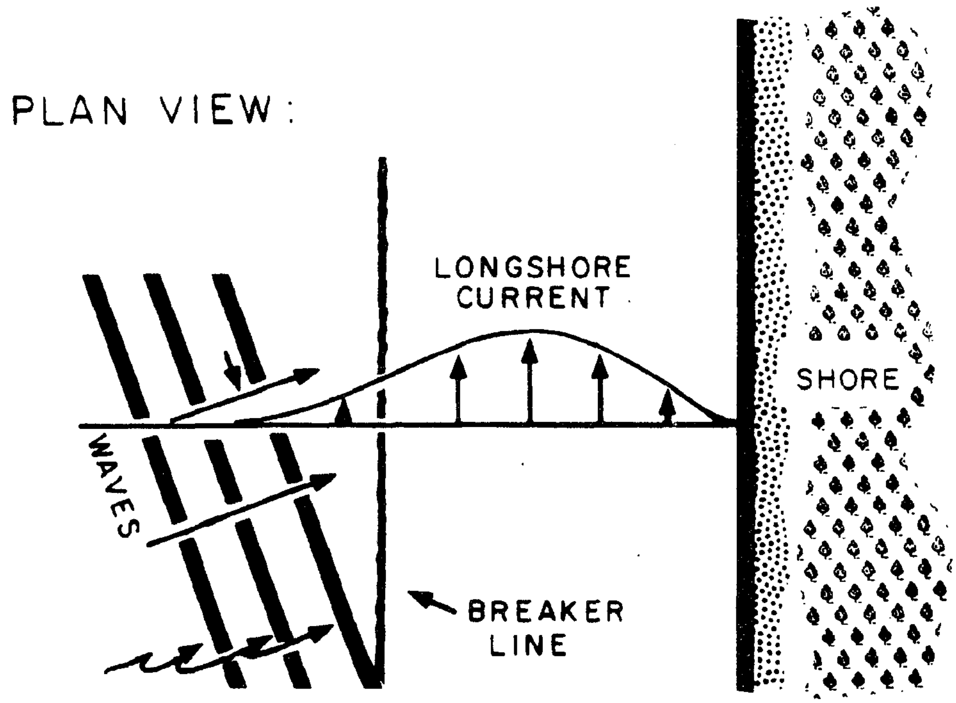
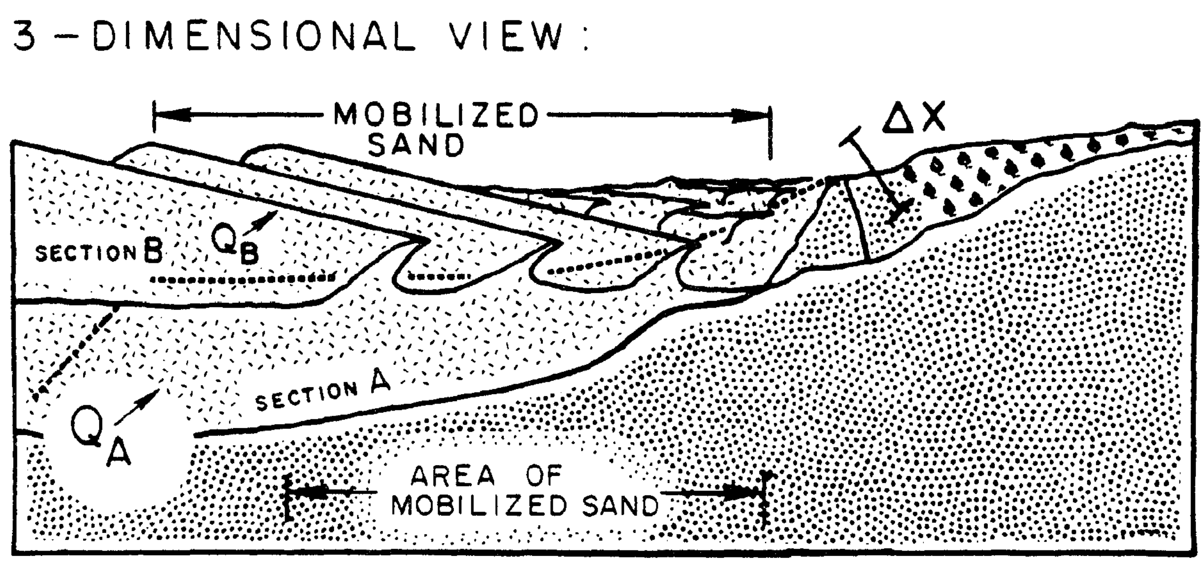


Figure 5.15



trapping effect of the groin is "Q_N" and there will be a deposit on the up-drift side of the groin and an erosion on the down-drift side of the groin. The rate at which sand is deposited on the up-drift side of the groin is "Q_N" and the rate at which sand is eroded on the down-drift side is "Q_N".

The second measure of longshore sediment transport is the gross sediment transport rate "Q_G":

$$Q_G=Q_+ + Q_- \qquad 5.13$$

representing the total amount of sand being transported past a point on a straight and parallel shoreline. This measure would be of importance in evaluating the potential trapping by an inlet. If the inlet traps all of the sediment moving toward it, the erosion of sediment on the left side is "Q_-" and the erosion of sediment on the right side of the inlet is "Q_+".

A convenient representation, developed by Walton and Dean (1973) is the littoral drift rose (Figure 5.16), by which the potential for net littoral drift Q_N can be represented for arbitrary shoreline orientations given the wave information (wave direction, height, period and percentage of occurrence; all of which are available from the wind-wave model). The littoral drift rose is computed by selecting a shoreline orientation and then, for each wind direction and the various wind speeds and associated percentages, computing the littoral drift(ie. the predicted rate of longshore sediment transport) for that shoreline. Then a different shoreline orientation is selected and the process is repeated. Once a sufficient number of

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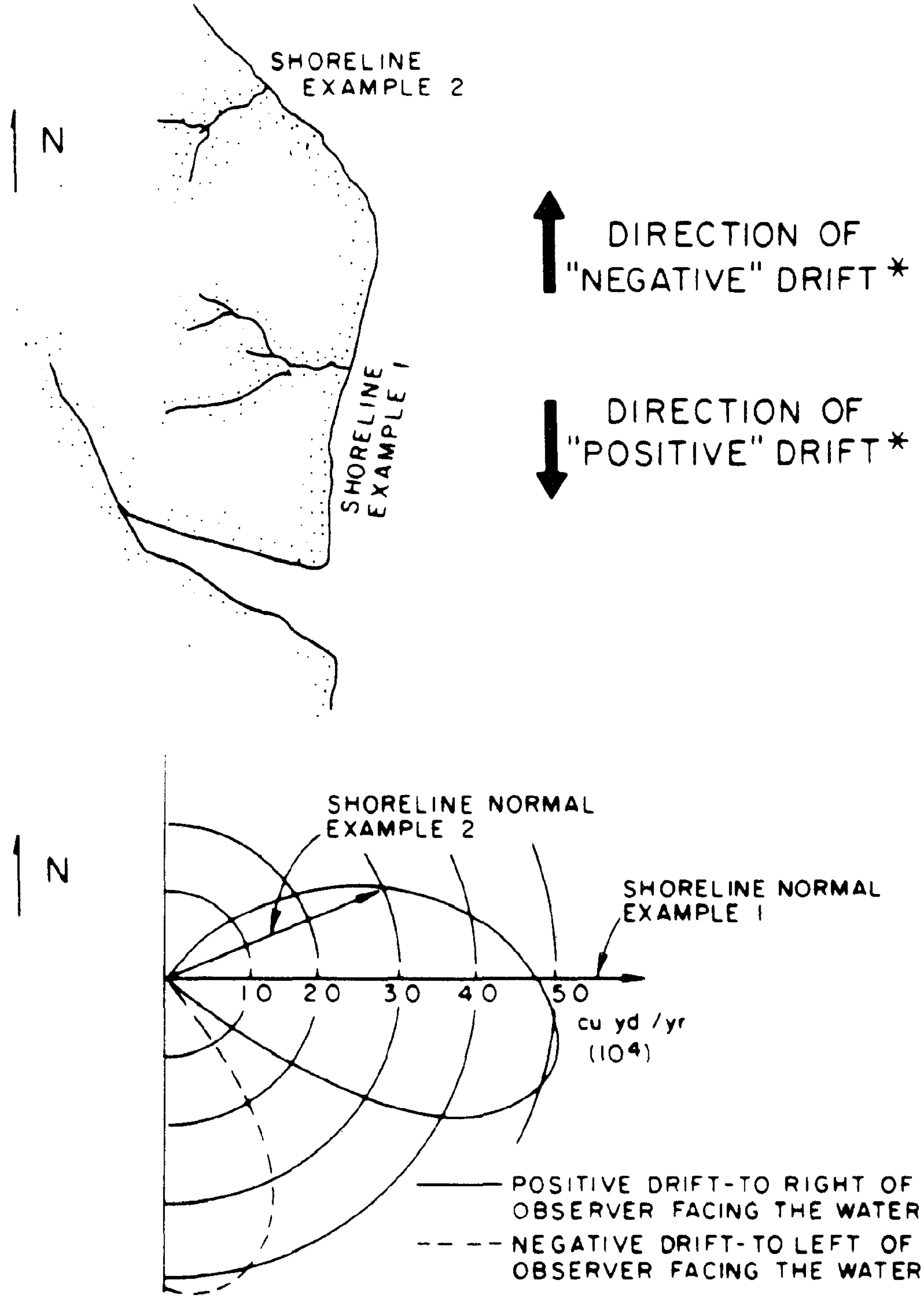
Opposite: Figure 5.16. Example applications of the littoral drift rose for two different shoreline locations.

Littoral drift roses indicate potential longshore sediment movement from computer simulation not verified by field data.

Figure 5.16

APPLICATIONS OF THE ROSE SHOWING POTENTIAL LITTORAL DRIFT RATES



* "POSITIVE" AND "NEGATIVE" ARE ARBITRARY DIRECTION DESIGNATIONS FOR REFERENCE USE ONLY

potential shoreline orientations is computed, the littoral drift rose may be drawn, which graphically represents the potential littoral drift versus shoreline orientation.

Figure 5.16 illustrates the use of a littoral drift rose for a hypothetical case for two different shoreline orientations. The solid lines of littoral drift represent "potential" transport (to the right as an observer faces the water) and the dashed lines represent "negative" transport (to the left). The potential annual rate of littoral drift is obtained by constructing a perpendicular to the shoreline and the value where it crosses either a solid or a dashed line represents the net annual littoral drift rate. In the case of a north-south shoreline (Example 1), the net annual littoral drift is approximately 47,000 cubic yards per year to the south. For Example 2, in which the shoreline is oriented north-northwest by south-southwest, the shoreline normal would be as shown and the net annual littoral drift would be 30,000 cubic yards to the right.

For this study, the potential rates of littoral drift were estimated from the computer models of wind and wave conditions which were described previously. The complete distribution of potential littoral drift roses in the northern Chesapeake Bay are plotted graphically in atlas form and have been supplied to the Maryland Department of Natural Resources. An example of the map atlas product is shown in Figure 5.3. Some idea of the kind of longshore movement of sediment which is predicted around the upper Bay is

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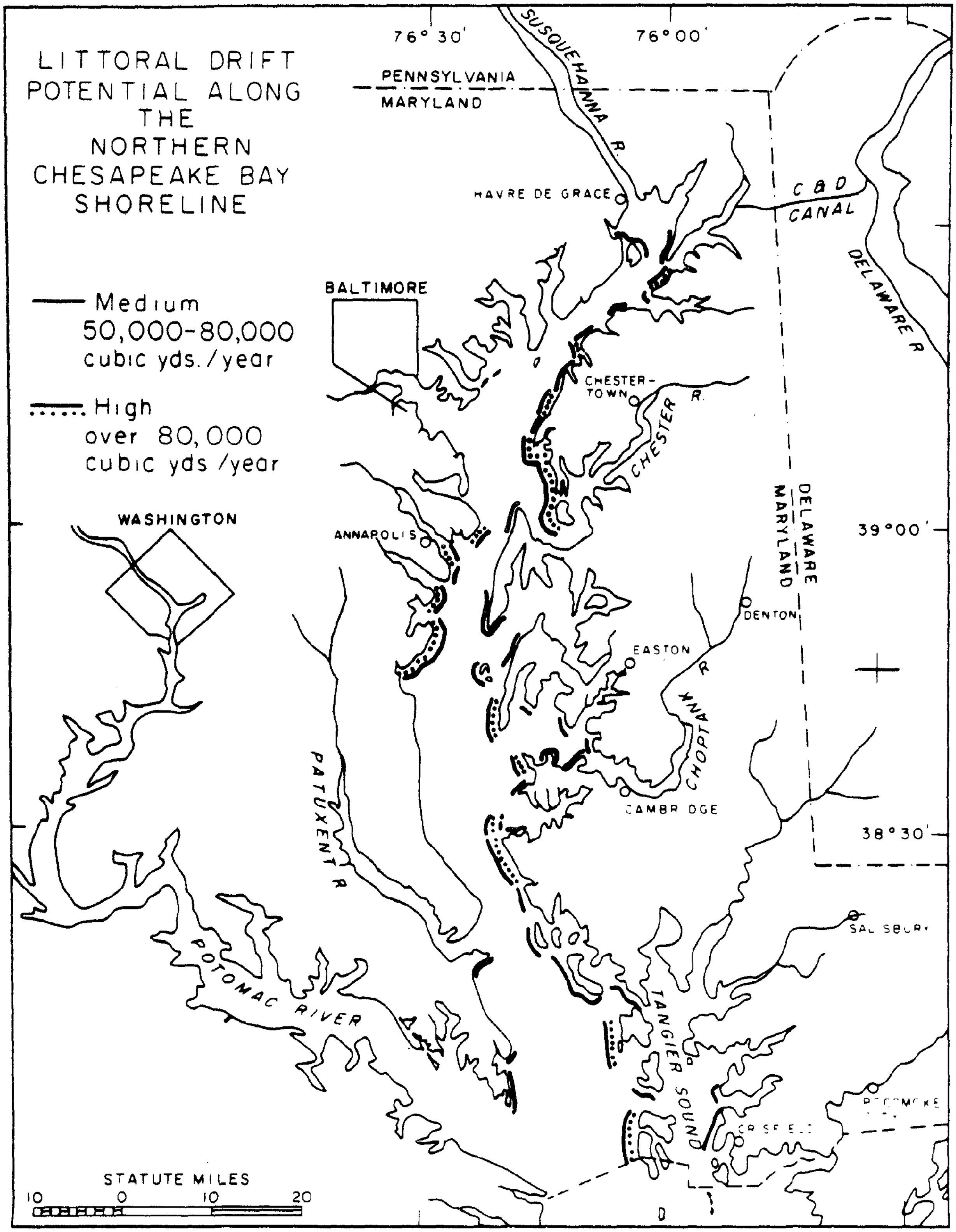
Opposite: Figure 5.17. Map showing potential littoral drift of shoreline sediments on northern Chesapeake Bay shoreline.

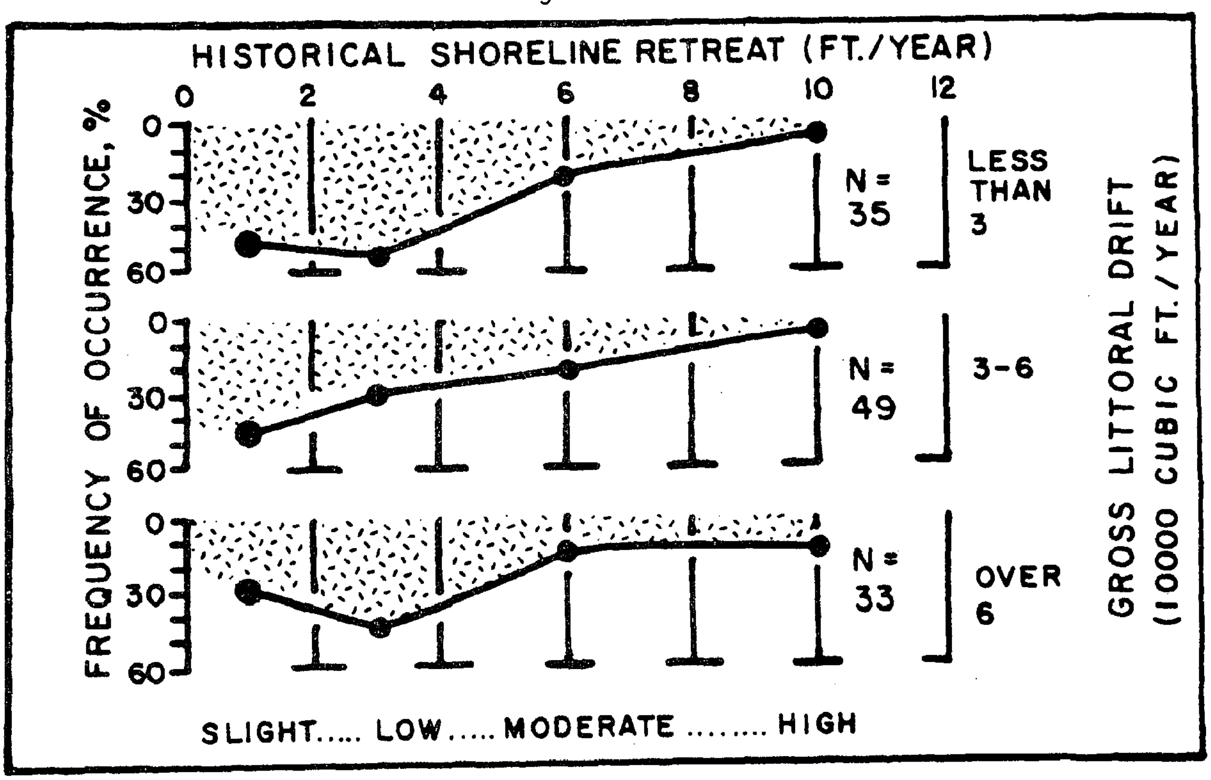
Figure 5.17


LITTORAL DRIFT POTENTIAL ALONG THE NORTHERN CHESAPEAKE BAY SHORELINE
Medium 50,000-80,000 cubic yds./year
High over 80,000 cubic yds /year
76° 30'
76° 00'
39°00'
38°30'
PENNSYLVANIA
MARYLAND
DELAWARE
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C & D CANAL
DELAWARE R.
BALTIMORE
CHESTER-TOWN
CHESTER R.
WASHINGTON
ANNAPOLIS
DENTON
EASTON
CHOPTANK R.
CAMBRIDGE
PATUXENT R.
POTOMAC RIVER
TANGIER SOUND
STATUTE MILES
10 0 10 20

illustrated in the map in Figure 5.17. The littoral drift is categorized as "medium" if net littoral transport past a fixed point is estimated between 50,000-80,000 cubic yards/year, and as "high" if the net transport is estimated at greater than 80,000 cubic yards per year. It is important to note that these numerical estimates apply to the rates of longshore movement of all sediment which can be moved about by waves. This can be considered to include sediments at least out to around the nine foot bathymetric contour.

All the highly-eroding reaches illustrated in Figure 5.2 can be compared with the distribution of potential littoral drift rates in

Figure 5.18



Above: Figure 5.18. Graph of relationship between the rate of coastal retreat and the distribution of potential littoral drift rates for northern Chesapeake Bay.

Figure 5.19. This comparison shows most of the highly-eroding areas are situated in zones with potentially high rates of longshore transport of sediments. This is to be expected, since longshore transport of sediments results from a predominance of waves approaching the shoreline from an angle, and the results will be movement of the eroded shoreline sediments away from erosion sites. However, there are reaches with "high" potential net littoral drift which are not highly eroding.

After deriving the computer estimates of the distribution of littoral drift, the variations in predicted gross rates of littoral drift were compared to historic erosion rates for all reaches at least 0.5 km. long which contained a uniform prediction of littoral drift characteristics and a uniform historic erosion rate. The results are shown in Figure 5.18. As in the other comparisons discussed so far in this chapter, most of the reaches which were suitable for analysis have "low" rates of erosion.

The similarities in the curves show there are no general differences in the way potential littoral drift rates are distributed between reaches with low, medium, or high historic erosion rates.

I. Relation of Rainfall to Coastal Retreat

On an annual basis, or even on a monthly basis, the amount of rainfall is rather uniform over the entire upper Chesapeake Bay. Table 5.4 summarizes the normals for the studied area. The total annual amount of roughly 44 inches is more or less evenly distributed over the year with the highest rate of approximately 4.5 inches/month occuring in July and August on the high end and the lowest rate of 3 inches/month occuring in the months of November and December. The spatial distribution of annual rainfall in the northern Chesapeake Bay region is shown in Figure 5.19. This map can be compared with the distribution of highly eroding areas in Figure 5.2 to show the relationship of rainfall to coastal retreat. But, there appears to be no substantial differences in the distribution of total annual rainfall which could help to explain the variation in shore erosion in the northern Chesapeake Bay over long periods of time.

Opposite: Table 5.4. Monthly rainfall data between 1931 and 1960 for Maryland.

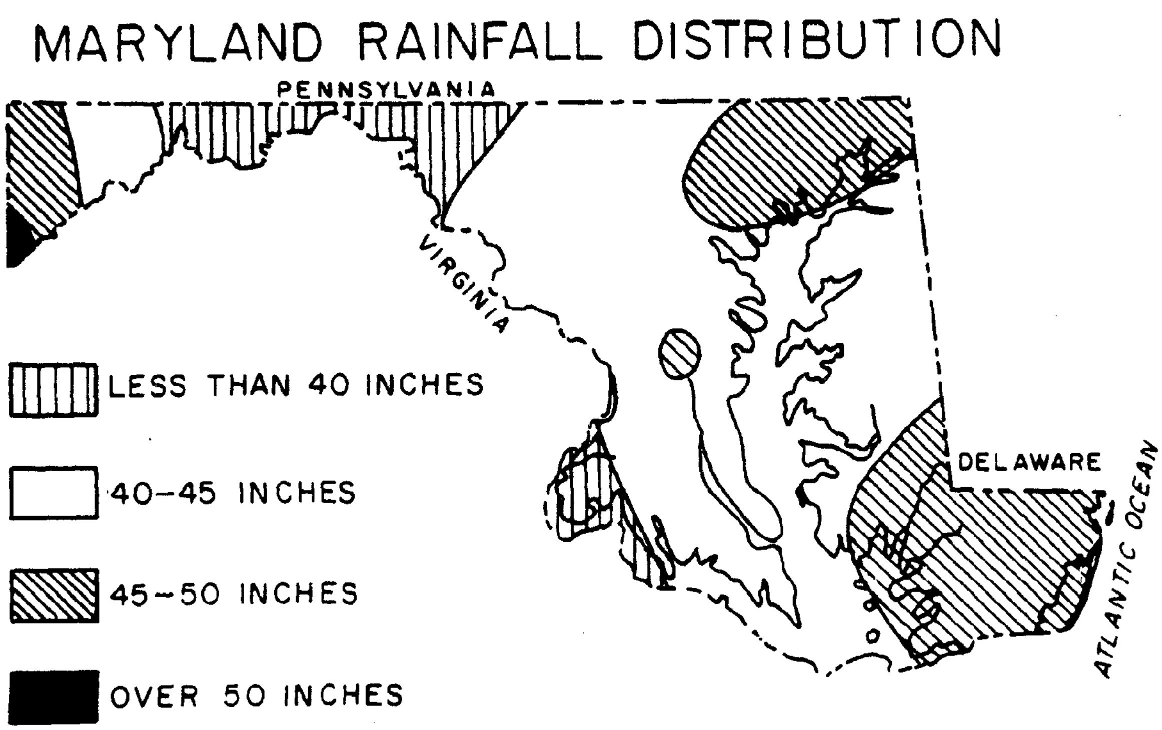
Figure 5.19 Spatial distribution for the average rainfall in Maryland, from Walker, 1970.

Table 5.4

Rainfall Data - Monthly Normals - Years 1931 - 1960 (inches)

Station	Jan	Feb	Mar	Apr	May	June	July	Aug	Sep	Oct	Nov	Dec	Annual
Elkton, MD	3.46	2.99	4.19	3.60	4.25	3.96	4.35	5.02	3.56	3.23	3.55	3.19	45.35
Annapolis, MD	3.14	2.57	3.62	3.31	3.83	3.51	4.14	4.50	3.46	2.63	2.78	2.85	40.34
Crisfield, MD	3.56	3.15	4.01	3.66	3.69	3.31	5.05	5.05	3.83	3.37	3.24	2.92	44.89
Baltimore, MD	3.43	2.89	3.82	3.60	3.98	3.29	4.22	5.19	3.33	3.18	3.13	2.99	43.05
Coleman, MD	3.61	2.93	3.86	3.43	4.17	3.64	4.29	4.97	3.17	3.08	3.41	3.18	44.28
Solomons, MD	3.55	2.78	3.61	3.50	3.76	3.45	5.57	5.00	3.59	3.11	3.33	2.97	44.22
Washington, D.C.	3.03	2.47	3.21	3.15	4.14	3.21	4.15	4.90	3.83	3.07	2.84	2.78	40.78

Figure 5.19



J. Characteristics of Highly-Eroding Reaches

The previous sections of this chapter have described the information on coastal processes which was compiled for this study, and have examined the relationship of each individual factor to the distribution of historic erosion rates found on different reaches in the northern Chesapeake Bay. Except for shoreline terrain, there were no clear differences which were able to be illustrated between any of the characteristics discussed (waves, tides, storm surges, potential littoral drift rates, or rainfall) and the historic erosion rates around the northern Bay. The relationship between shoreline terrain and historic erosion in Figure 5.4 shows that reaches with the highest historic erosion rates generally are composed of banks less than 10 feet high. A few reaches with high historic erosion rates were also found to consist of marsh. But Figure 5.4 also shows that reaches with marsh, or with higher shoreline banks or bluffs, generally possess low historic rates of coastal retreat.

The same type of comparison between historic erosion rates and wave climate, tide range, storm surge, or littoral drift rates failed to illustrate any important differences in the ways each of these characteristics is individually related to the historic erosion of different reaches around the northern Chesapeake Bay.

These are the results of one type of approach which can be taken towards evaluating shore erosion in the northern Chesapeake Bay i.e., by first producing maps of predicted wave climate, tide range, or storm surge characteristics around the Bay margins, then next selecting reaches at least 0.5 km. in length where both the historic erosion rate, and any particular prediction, are uniform and both are able to be characterized by a single value, and finally assessing whether reaches with different erosion rates are distributed any differently for one class (for instance, of wave climate or shoreline terrain) than for any other.

The results of this approach to evaluating shore erosion were not presented along with any measure of statistical significance, since the statistical meaning would be difficult to interpret. This is because the classifications used in the previous sections represented only the gross characteristics of the shoreline within any reach, and considerable variations can and do exist within each reach. Therefore, the type of evaluation contained in the previous sections is somewhat subjective at places where the shoreline variations are irregular within a reach. The results of the analysis in the previous sections simply illustrate that qualitatively, none of the factors which are part of the erosion process are individually sufficient to explain why some shoreline reaches have eroded at high rates, while others have retreated more slowly.

An alternate approach to evaluating shore erosion in the northern Bay consists of defining <u>fixed</u> reaches (see Figure 5.20) and categorizing shoreline characteristics within each reach. Again, this approach is somewhat subjective, since shoreline variations are irregular within each reach. In particular, it seems important to point out that no generalizations about the shore erosion process in the northern Chesapeake Bay should probably be applied to specific shoreline lengths any less than 0.5 km. long. But within this broad definition, some generalizations can be made about the 32 highly-eroding reaches mentioned on pages 5-4 and 5-6. The generalizations listed below are drawn from the information on the maps presented in the previous sections, and from the reach categorizations in Table 5.6 at the end of this chapter.

1. All of the thirty-two highly-eroding reaches are located in regions of either Lowland Deposits or Quarternary Deposits Undivided. The material represented by these geological classifications is, however, quite non-uniform and some formations in outcrop should actually have high resistance to erosion. Furthermore, not all of the shoreline reaches composed of these geological formations are subject to high-erosion rates. For instance, the bayside of Kent Island (reaches 75 to 77) is highly erosional yet the adjacent shoreline in Queen Annes County north of the Chester River experiences only slight shoreline retreat although both regions have similar geological and environmental conditions.
2. All the thirty-two reaches are in areas of low shoreline relief. All the nine high-erosion reaches are points, islands or tips of lowland.
3. Most of the thirty-two reaches, except one (reach 80) on the eastern shore, are situated in medium-to-high wave-energy zones. Again, the reverse is not true. The shore-

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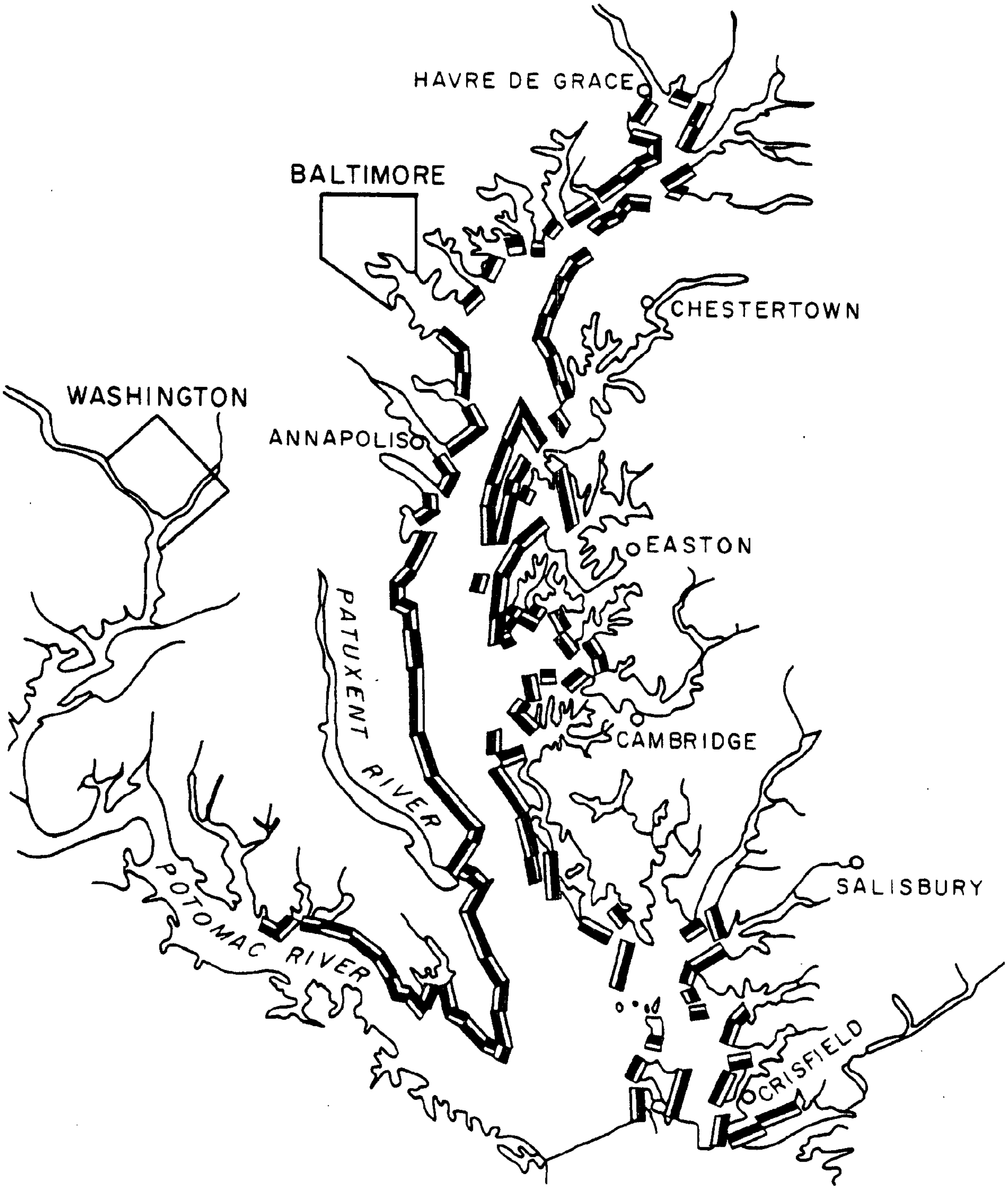
<u>Opposite:</u> Figure 5.20. Map of northern Chesapeake Bay shoreline reach designations. General characteristics of each reach are presented in Table 5.6.

Figure 5.20

Illustration of Northern Chesapeake Bay
Reach Designations Shown in Table 5.6



line along Calvert County, for instance, is in the high wave-energy zone. Yet, the historic rate of shoreline retreat is small.

4. The correlation between "net littoral drift" and "historical erosion" is less coherent. Reaches 64, 71, 72 and 73 do not currently have high erosion, although the net drift values for these reaches are high. Thus, the maximum value in the littoral drift rose derived for any particular reach provides a better indication of erosion potential than the net drift for a certain shore orientation within any reach.

A more detailed investigation was performed for the nine high-erosion reaches to see whether common factors could be identified.

Reach 14 - Pt. Lookout to Saint Jerome Creek, St. Mary's County

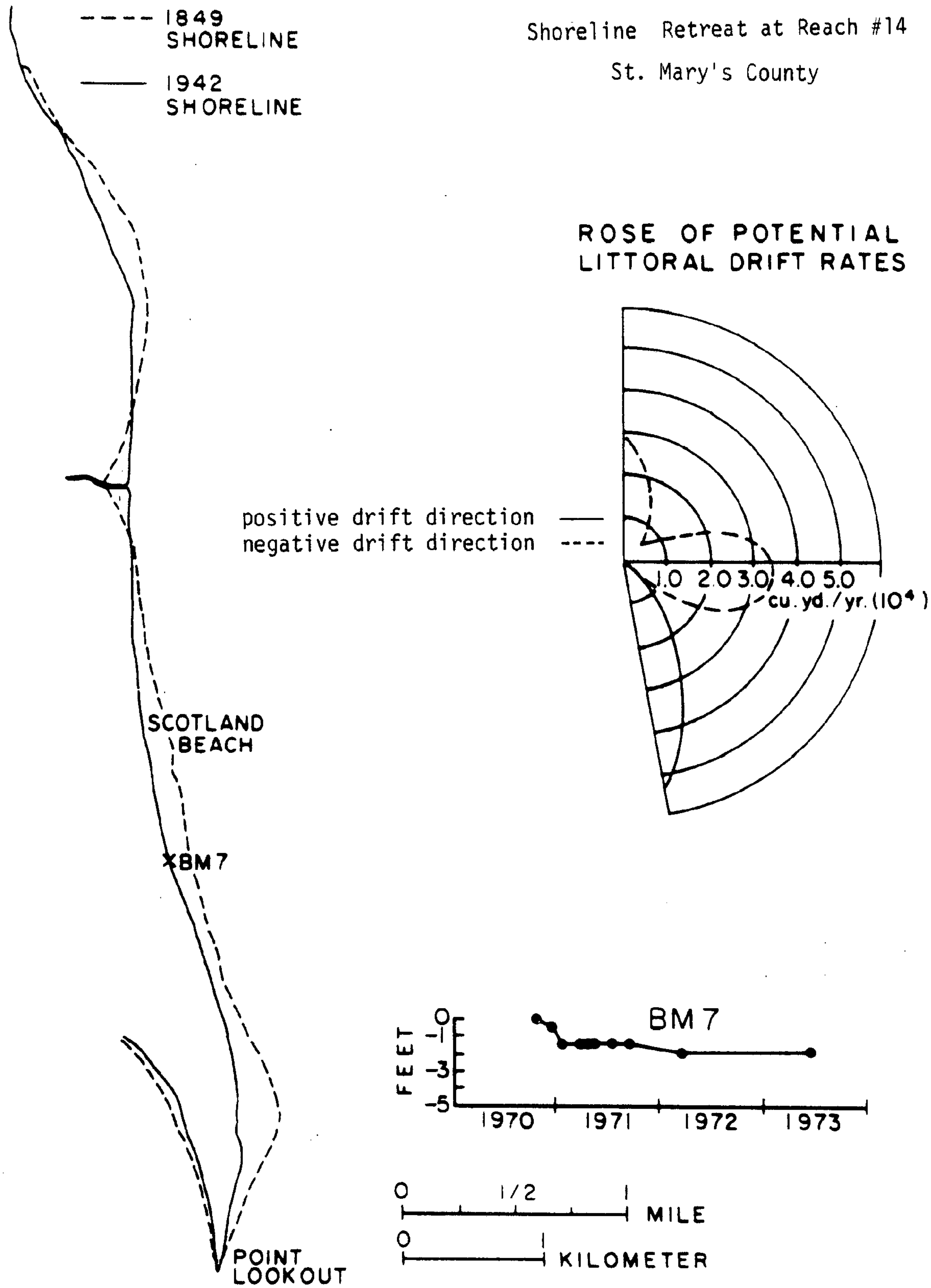
This reach is about 5 miles long at the southern tip of St. Mary's County where the Potomac River meets the Chesapeake Bay. According to a historical map from 1849-1942, the shoreline retreat has been quite significant. There are three sub-reaches (about 1/3 of the total length) where the erosion can be classified as high (see Figure 5.21). Recent surveys at the control line "BM7", however, seemed to indicate that the erosion has decreased considerably in this vicinity to less than 1 ft/yr. These survey results could be misleading as they were taken at the updrift end of coastal protection structures emplaced at Scotland Beach.

Opposite: Figure 5.21. Shoreline retreat at Reach 14.

Figure 5.21

Shoreline Retreat at Reach #14

St. Mary's County



Geologically, this area is completely in the Quaternary Lowland Deposits with low coastal relief. The sub-aerial material is non-uniform but is generally of high resistance to erosion. The offshore slope is mild with fine sand found in the center section of the reach (see Case No. 13 structure described in Chapter II). Also, there is no material supply from the south other than from offshore.

The area is exposed to a long fetch from the south, and is thus susceptible to tropical storm attack. The annual wave energy as computed is classified as "high".

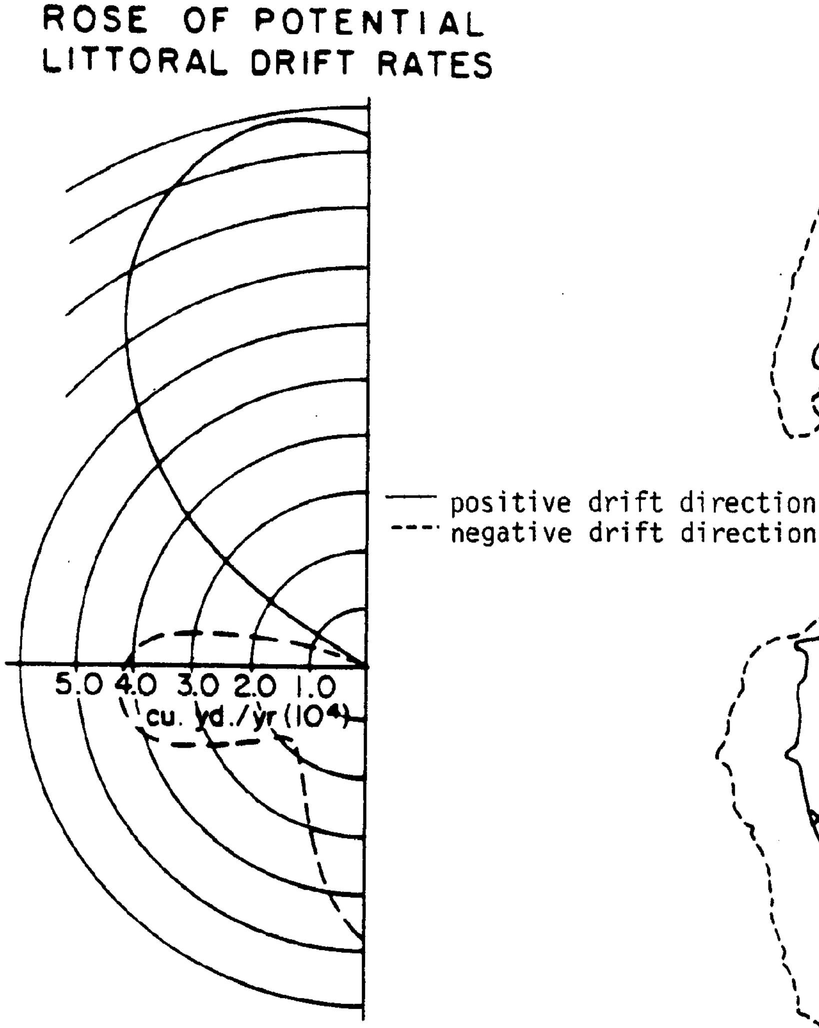
The predicted annual littoral drift is predominantly toward the north at approximately 3 to 4 x 10^4 cubic yards per year. This quantity is considered to be "medium". The fact that the drift is toward the north (with no material from the south), coupled with low sub-aerial relief, results in high shoreline retreat.

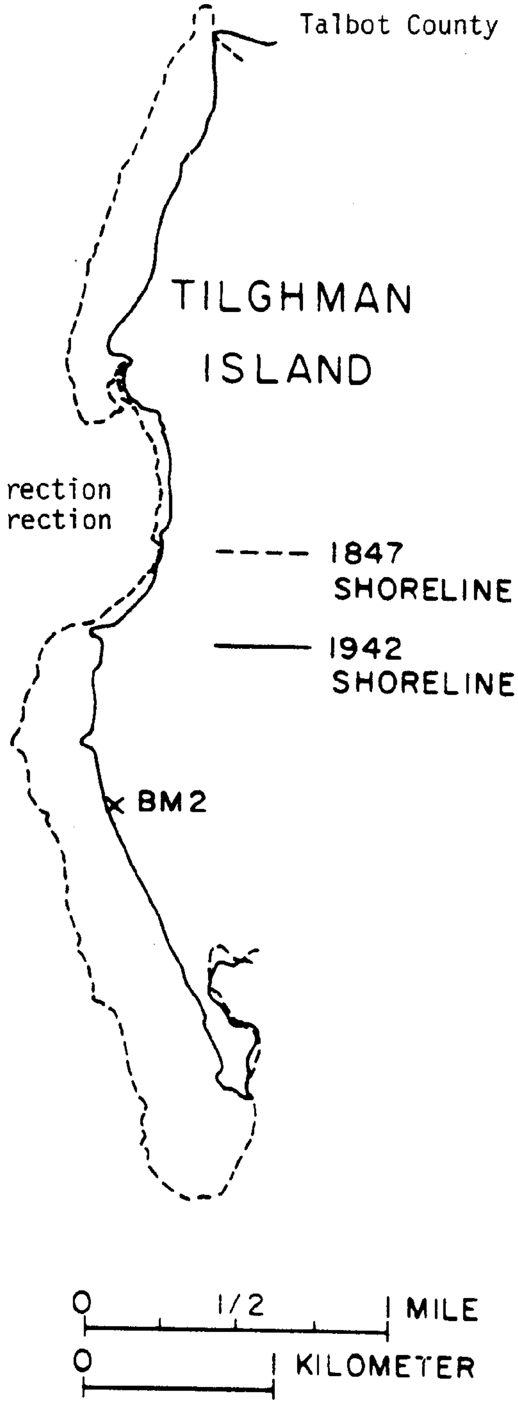
Reaches 26 and 31, Holland Point and Thomas Point, Anne Arundel County

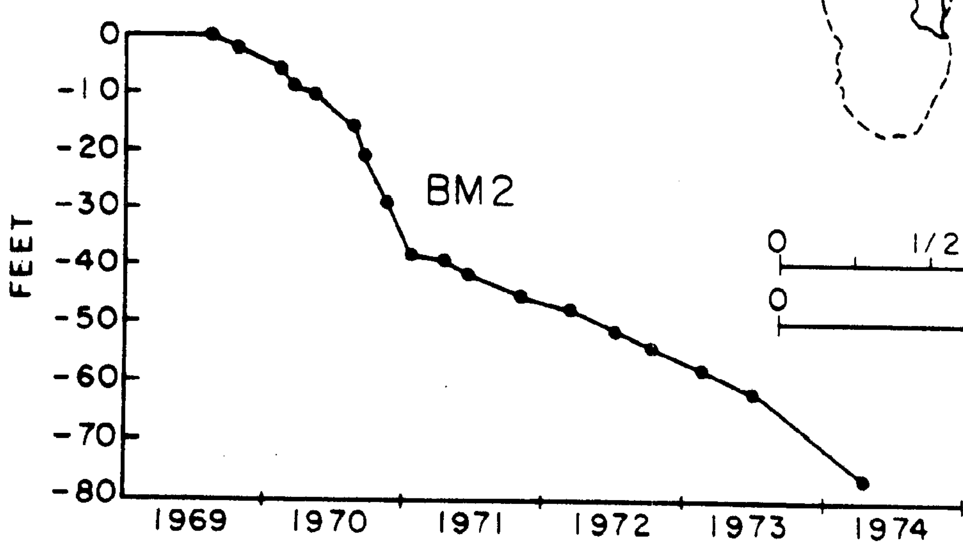
The high erosion is a local effect near the points. Geologically the areas are in Lowland Deposits with low sub-aerial relief. The material has high resistance to erosion. This is a high wave energy zone and exposed to both tropical and extratropical storm attack. The orientation of the point also makes it vunerable to high erosion. The predictions of littoral drift indicate that material moves away from these points in both direction.

Opposite: Figure 5.22. Shoreline retreat at Reach 86.

Figure 5.22
Shoreline Retreat at Reach # 86
Talbot County







Reach 77, Price Creek to Kent Point, Kent Island, Queen Annes County

This reach is the southern part of Kent Island. The erosion rate is actually between medium to high but becomes progressively more severe near the southern tip.

Geologically, it is Quaternary Lowland Deposits with low relief. The sub-aerial material is non-uniform but generally can be classified as low-to-medium resistance to erosion. The wave energy level is high; the erosion potential is also high. In addition, the littoral drift direction is unfavorable in that there is no material supply from the adjacent land. In general, the combined conditions are conducive for causing a high erosion rate.

Reach 86 - Lowes Point to Knapps Narrows, Talbot County

This reach is the southern part of Tilghman Island. The area suffers high erosion except for a short sub-reach in the middle where the erosion is low (Figure 5.22). Based upon recent surveys, the erosion has not been slowing down; 80 linear feet have been lost during a recent five year period (Figure 5.22).

This area is also composed of Quarternary Lowland Deposits with low sub-aerial relief. The material is very non-uniform varying from low resistance to high resistance. Because the area is flanked by shallow shoals, the wave energy is only moderate, although the area is exposed. Based upon the littoral drift rose and the present shoreline

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Opposite: Table 5.5. Storm wave conditions at the nine high-erosion reaches.

Table 5.5

Storm Wave Conditions at the Nine High-Erosion Reaches

	Storm Wave Height (ft)					
Reach	1 year*	5 year*	10 year**	20 year**	50 year**	100 year**
14	5.56	8.83	10.31	11.11	12.20	12.76
26	5.20	8.25	9.85	10.66	12.11	13.03
31	0.50	0.79	9.14	9.97	11.00	11.67
77	3.86	6.15	8.16	8.88	10.13	10.92
86	4.53	6.92	7.63	8.57	9.38	9.95
87	4.55	7.17	7.57	8.39	9.32	9.99
99	5.30	8.18	3.89	4.57	4.96	5.25
101	4.93	7.04	7.18	7.80	8.84	9.58
103	4.59	6.73	8.68	9.43	10.51	11.29
104	4.74	7.15	7.84	8.48	9.57	10.23

*Based upon extra-tropical storm (Northeaster). Main direction from north, NE or NW.

**Based upon tropical storms. Main direction from south, SW or SE.

orientation, the littoral drift should be moderate; however it can account for the high shoreline retreat because of the low shoreline relief.

Reaches 99, 101, 103 and 105 - From Mills Point to Barren Island, Dorchester County

All these reaches are in the Quaternary Deposits Undivided area, all with very low relief of only a few feet. The sub-aerial material is generally considered highly resistant to erosion. All the sub-reaches that experience high historical erosion are on the bayside. All of them are either islands or tips of lowland protruding into the bay.

Because of shallow water conditions surrounding these reaches, the annual wave conditions are not exceptionally high compared with the rest of the bayshore. However, the storm waves could be just as severe as any exposed area because of high storm surge conditions. All the reaches except Reach 99 are susceptible to both extra-tropical and tropical storm waves. Reach 99, on the other hand, is sheltered from severe waves from the south, but is subject to higher waves from the north than the other reaches. Almost all of them are subject to high potential littoral drift but not necessarily high net drift with their present shoreline orientations.

Table 5.5 summarizes the storm wave conditions at the nine high-erosion reaches. In this table, the "1-year" and "5-year" storm waves are based upon extra-tropical storms with directions assumed to be from N, NE or NW; abnormally low values (Reach 31) mean the reaches are shielded from this type of storm. The storm waves with return period higher than 10 years are based upon tropical storms with direction from S, SE or SW; again, exceptionally low values (Reach 99) result from reaches shielded from tropical storm waves.

K. Classification of Coastal Characteristics

For future assessment of erosion conditions in northern Chesapeake Bay, Table 5.6 was prepared. In this table, the shoreline is identified by reaches shown in Figure 5.20; within each reach, the historical erosion rate is listed along with many other important factors. The interpretations of each column are given here.

Column 1: Reach Number - See Figure 5.20 for location

Column 2: Historical Erosion Rate - Data from MCZMP atlas "Historical Shorelines and Erosion Rates."

- S (Slight) - less than 2 linear feet per year
- L (Light) - 2 to 4 linear feet per year
- M (Medium) - 4 to 8 linear feet per year
- H (High) - greater than 8 linear feet per year
- A (Accretion) - Accretion reach

Column 3: Shoreline Characteristics

Dominant Type: From page 5-7

Column 4: Mean Tide Range - In feet from Appendix B

Column 5: 100-Year Storm Surge - In feet from Appendix B

Column 6: Wave Energy: Based on maximum wave height in 30 mph wind.

- L - less than 2.5 ft.
- M - 2.5 ft. to 4 ft.
- H - higher than 4 ft.

Column 7: <u>Net Drift Characteristics</u>

<u>Drift Potential</u>: (in 10^4 cy/yr.) Based on the net littoral drift rose and mean shoreline orientation as explained in Section H.

+ means drift direction towards right with observer facing the water.

- means drift direction towards left with observer facing the water.

x means difficult to interpret.

Double value means two major shoreline orientations in one reach.

The drift directions analyzed from aerial photographs are included in Table 5.6 to aid in the determination of the stability and direction of littoral drift as well as to compare with the prediction. As can be seen from the comparison, the agreement between the predicted and observed drift directions is quite good.

It must be noted that the historic erosion rate classifications in Table 5.6 represent only the gross characteristics within each reach. Considerable variations could exist within each reach. That is, if a reach is classified as highly erosional, it may contain sub-regions where the erosion is slight or even accretional. Therefore, the classification is somewhat subjective at times when the shoreline variations are irregular within the reach.

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#1 POTOMAC RIVER—ST. CATHERINE IS. TO COLTONS POINT.	5	2	—	—	M	X (+,-)	ST. MARYS
#2 POTOMAC RIVER - COLTON'S POINT— CANOE NECK CREEK	—	2	—	—	M	-10,000 CU. yds/YR. ( )	ST. MARYS
#3 POTOMAC RIVER NEWTOWN NECK	—	3	—	—	M	0 ( )	ST. MARYS
#4 POTOMAC RIVER HUGGINS PT - FLOOD CREEK	—	3	—	—	M	0 (+)	ST. MARYS
#5 POTOMAC RIVER FLOOD CREEK — HERRING CREEK	5	2	—	—	M	0 TO -15,000 CU YDS./YR (-)	ST. MARYS
#6 POTOMAC RIVER HERRING CREEK — PINEY POINT	L	2	—	—	M	-40,000 CU. YDS./YR (-)	ST. MARYS
#7 POTOMAC RIVER PINEY POINT— STRAITS POINT	A	2	—	—	M	0 ( )	ST. MARYS
#8 POTOMAC RIVER ST. GEORGES ISLAND	M	2	—	—	M	+4000 CU. YDS./YR ( )	ST. MARYS
#9 ST. MARYS RIVER EDMUND PT. — ST. GEORGES ISLAND	5	2	—	—	M	-25,000 CU. YDS./YR. ( )	ST. MARYS

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#10 ST. MARY'S RIVER PRIEST POINT — KITTS POINT	L	3	—	—	M	30,000 CU. YDS./YR ( )	ST. MARY'S
#11 POTOMAC RIVER KITTS POINT - GRAY POINT	M	3	—	—	M	0 ( )	ST. MARY'S
#12 POTOMAC RIVER GRAY POINT - CORNFIELD POINT	L	2	—	—	M	0 ( )	ST. MARY'S
#13 POTOMAC RIVER CORNFIELD POINT - POINT LOOKOUT	S	2	—	—	M	0 ( )	ST. MARY'S
#14 POINT LOOKOUT - ST. JEROME CREEK	H	2	1.0	4.7	H	-32,000 CU. YDS./YR ( )	ST. MARY'S
#15 ST. JEROME CREEK - POINT NORTH POINT	L	2	1.0	4.6	H	-35,000 CU. YDS./YR ( )	ST. MARY'S
#16 POINT NORTH POINT - PINE HILL RUN	M	2	1.0	5.0	H	+4000 CU. YDS./YR (+)	ST. MARY'S
#17 PINE HILL RUN - CEDAR POINT	S	2	0.9	5.4	H	-40,000 CU. YDS./YR. (+, -)	ST. MARY'S
#18 CEDAR POINT - HOG POINT	M	2	1.0	5.4	H	0 ( )	ST. MARY'S

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#19 DRUM POINT – LITTLE COVE POINT	S	6	1.0	5.4	H	-10,000 CU.YDS./YR (+)	CALVERT
#20 LITTLE COVE POINT- COVE POINT	L	6	1.2	5.2	H	-35,000 CU.YDS./YR ( )	CALVERT
#21 COVE POINT- LONG BEACH	L	6	1.1	5.1	H	+35,000 CU.YDS./YR ( )	CALVERT
#22 LONG BEACH – PARKER CREEK	S	6	0.9	4.9	H	+30,000 CU.YDS./YR ( )	CALVERT
#23 PARKER CREEK- PLUM POINT	L	4	0.9	5.2	H	0 ( )	CALVERT
#24 PLUM POINT- CHESAPEAKE BEACH	S	6	0.9	5.9	H	2000 CU.YDS./YR ( )	CALVERT
#25 CHESAPEAKE BEACH- HOLLAND POINT	M	2	0.9	6.2	H	0 ( )	ANNE ARUNDEL
#26 HOLLAND POINT	H	2	0.9	6.3	H	— (-)	ANNE ARUNDEL
#27 HERRING BAY	S	6	0.9	6.3	H	-4,000 CU.YDS./YR (+,-)	ANNE ARUNDEL

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#28 ROCK HOLD CREEK – BROADWATER CREEK	M	5	0.9	6.6	H	-80,000 CU.YDS./YR ( )	ANNE ARUNDEL
#29 BROADWATER CREEK – CURTIS POINT	M	5	0.9	6.9	H	-70,000 CU.YDS/YR. (–)	ANNE ARUNDEL
#30 DUTCHMAN POINT – TURKEY POINT	L	5	0.9	7.0	M	-90,000 TO -10,000 CU.YDS./YR (–,+)	ANNE ARUNDEL
#31 THOMAS POINT	H	5	0.9	7.0	H	+15,000 CU.YDS./YR (+)	ANNE ARUNDEL
#32 THOMAS POINT – TOLLY POINT	L	5	0.8	7.0	M	0 (–)	ANNE ARUNDEL
#33 TOLLY POINT – CHINKS POINT	S	5	0.8	7.1	M	-55,000 CU.YDS./YR (–)	ANNE ARUNDEL
#34 HACKETT POINT – SANDY POINT	L	5	0.8	7.1	H	-90,000 CU.YDS/YR (+)	ANNE ARUNDEL
#35 SANDY POINT – PERSIMMON POINT	S	3	0.9	7.1	H	-2,000 TO +15,000 CU.YDS./YR (–,+)	ANNE ARUNDEL
#36 MOUNTAIN POINT – GIBSON ISLAND BEACH	L	2	0.9	7.2	H	+20,000 CU.YDS./YR (+)	ANNE ARUNDEL

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#37 GIBSON ISLAND BEACH – BODKIN POINT	5	5	0.9	7.5	H	+10,000 CU.YDS./YR (+)	ANNE ARUNDEL
#38 BODKIN POINT – ROCK POINT	L	5	1.0	7.8	H	-20,000 CU.YDS/YR (-)	ANNE ARUNDEL
#39 NORTH POINT – BAYSHORE PARK	L	1-7	1.0	8.1	H	0 ( )	BALTIMORE
#40 BAYSHORE PARK – BAY ISLAND BEACH	L	1-9	1.1	8.7	H	0 ( )	BALTIMORE
#41 HART ISLAND	L	1-9	1.2	8.9	H	-20,000 CU.YDS/YR ( )	BALTIMORE
#42 WELLS POINT – HOLLY BEACH	5	3	1.2	9.5	M	0 (+,-)	BALTIMORE
#43 MIAMI BEACH	5	1	1.4	9.8	M	-10,000 CU.YDS/YR (-)	BALTIMORE
#44 RICKETT POINT – ROBINS POINT	L	2-9	1.6	12.1	H	-35,000 CU.YDS/YR ( )	HARFORD
#45 FORDS POINT – LEGES POINT	L	2-9	1.5	9.2	M	-15,000 CU.YDS/YR ( )	HARFORD

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#46 ABBEY POINT - LOCUST POINT	L	2.9	1.4	9.3	M	-20,000 CU.YDS/YR (+)	HARFORD
#47 TAYLOR ISLAND POINT - STONY POINT	M	1.7	1.5	10.2	M	0 (-)	HARFORD
#48 STONY POINT - BLACK POINT	L	2	1.5	10.8	M	-8000 CU.YDS/YR ( )	HARFORD
#49 BEAR POINT - SANDY POINT	L	1	1.6	10.9	M	-8000 CU.YDS/YR ( )	HARFORD
#50 SANDY POINT - LOCUST POINT	S	1	1.6	11.0	L	+25000 CU.YDS/YR ( )	HARFORD
#51 LOCUST POINT - PLUM POINT	S	1	1.6	11.3	L	-30000 CU.YDS/YR ( )	HARFORD
#52 SWAN CREEK POINT - CONCORD POINT	S	3	1.7	11.5	L	-10 000 CU.YDS./YR (+,1)	HARFORD
#53 POPLAR POINT - CARPENTER POINT	S	3	1.7	11.5	L	0 ( )	CECIL
#54 RED POINT - ROCKY POINT	S	4	1.6	11.4	L	+10,000 CU.YDS/YR (+,-)	CECIL

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#55 ROCKY POINT - TURKEY POINT	S-A	4	1.6	10.9	L	0 (+)	CECIL
#56 TURKEY POINT	S	4	1.5	10.8	M	0 ( )	CECIL
#57 ROCK POINT - GROVE POINT	L	4-9	1.6	10.7	M	+66000 CU.YDS/YR ( )	CECIL
#58 GROVE POINT	L	4	1.6	10.6	M	0 ( )	CECIL
#59 BETTERTON - HOWELL POINT	S	4	1.6	10.7	M	17000 CU.YDS/YR (+,-)	KENT
#60 HOWELL POINT	S	2	1.5	10.0	M	+30,000 CU.YDS/YR (+)	KENT
#61 HOWELL POINT - MEEKS POINT	S	3	1.4	9.8	M	+38,000 CU.YDS/YR ( )	KENT
#62 MEEKS POINT - PLUM POINT	S-A	2-8	1.4	9.3	L	0 (-,+)	KENT
#63 PLUM POINT - WORTON POINT	S	2-7	1.4	9.2	M	+50000 CU.YDS/YR (+)	KENT

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#64 WORTON POINT - COPELAND	L	2-8	1.3	8.8	M	-70,000 CU.YDS/YR (-)	KENT
#65 HANDYS POINT - FAIRLEE CREEK	5-A	2-8	1.4	8.7	M	0 (-)	KENT
#66 MITCHELL BLUFF	5	4	1.5	8.6	M	50,000 CU.YDS/YR (-)	KENT
#67 TOLCHESTER BEACH	5-A	4	1.3	8.5	M	40,000 CU.YDS/YR (+,-)	KENT
#68 TOLCHESTER BEACH - TOWER 13	5-A	4	1.1	8.6	H	40,000 CU.YDS/YR (-)	KENT
#69 TOWER 13 - SWAN POINT	L	2	1.0	8.7	H	0 (-)	KENT
#70 SWAN POINT - WINDMILL POINT	L	3-8	1.0	8.4	M	-75,000 CU.YDS./YR (+)	KENT
#71 HUNTINGFIELD POINT - HICKORY THICKET	L	2	1.0	8.3	M	-90,000 CU.YDS/YR (-)	KENT
#72 HICKORY THICKET - WILSON POINT	5	2-9	1.0	8.0	H	-90,000 CU.YDS./YR (-)	KENT

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#73 WILKES BEACH	L	2-9	1.0	8.5	M	-85000 TO -10,000 CU.YDS/YR (-)	KENT
#74 CHESTER-LOVE POINT (KENT ISLAND)	L	3	1.0	8.2	M	10,000 CU.YDS/YR (+)	QUEEN ANNES
#75 LOVE POINT-MATAPEAKE (KENT ISLAND)	M	3	1.0	8.0	M	-15000 CU.YDS./YR (-)	QUEEN ANNES
#76 MATAPEAKE-CRANEY CREEK (KENT ISLAND)	S	3	1.0	7.1	H	0 (-)	QUEEN ANNES
#77 CRANEY CREEK-KENT POINT (KENT ISLAND)	M	2-5-8	1.1	7.0	H	-10,000 CU.YDS/YR (-)	QUEEN ANNES
#78 KENT POINT-LONG POINT (KENT ISLAND)	S	2-7	1.2	6.8	M	-70,000 CU.YDS/YR (-)	QUEEN ANNES
#79 LONG POINT-PHILPOTS ISLAND (KENT ISLAND)	L	3	1.2	6.7	M	-5000 CU.YDS/YR (+)	QUEEN ANNES
#80 PHILPOTS ISLAND-COX CREEK (KENT ISLAND)	M	3-8	1.2	6.7	M	0 ( )	QUEEN ANNES
#81 (EASTERN BAY) TURKEY POINT	S	3	1.2	6.7	M	25000 CU.YDS/YR ( )	QUEEN ANNES

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#82 BRIAN POINT – BENNETT POINT (EASTERN BAY)	L	3	1.3	6.7	M	0 ( )	QUEEN ANNES
#83 TILGHMAN POINT – WADES POINT (EASTERN BAY)	L	3	1.3	6.7	M	+20,000 CU. YDS/YR (+)	TALBOT
#84 WADES POINT – LOWES POINT (EASTERN BAY)	L	1	1.3	6.6	M	0 (+,–)	TALBOT
#85 LOWES POINT – KNAPPS NARROWS (TILGHMAN ISLAND)	M	5	1.3	6.4	M	0 ( )	TALBOT
#86 KNAPPS NARROWS – BLACK WALNUT POINT (TILGHMAN ISLAND)	H	5	1.5	6.4	M	–30,000 CU YDS./YR (+,–)	TALBOT
#87 POPLAR ISLAND	H	3-7	1.5	5.9	M	+20,000 CU YDS/YR ( )	TALBOT
#88 LOWER BAR NECK (TILGHMAN ISLAND)	L	5	1.5	5.9	M	–30,000 CU. YDS/YR ( )	TALBOT
#89 UPPER BAR NECK POINT – BALD EAGLE POINT (TILGHMAN ISLAND)	S	5	1.5	6.1	M	–20,000 CU. YDS/YR (–)	TALBOT
#90 CHANGE POINT – NELSON POINT (CHOPTANK RIVER)	M	5	1.5	6.1	M	+30,000 CU. YDS/YR (+)	TALBOT

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#91 NELSON POINT– BALLS CREEK (CHOPTANK RIVER)	L	5-8	1.5	6.1	M	-10,000 CU.YDS/YR ( )	TALBOT
#92 BRIDGE CREEK– IRISH CREEK (CHOPTANK RIVER)	L	5	1.5	6.1	M	+25,000 CU.YDS/YR (-)	TALBOT
#93 LUCY POINT– BENONI POINT (CHOPTANK RIVER)	M	3	1.5	6.1	M	-25,000 CU.YDS/YR (+)	TALBOT
#94 BACHLOR POINT– ISLAND CREEK (CHOPTANK RIVER)	S	2-9	1.6	6.1	M	-15,000 CU.YDS/YR (-)	TALBOT
#95 ISLAND CREEK– CHLORA POINT (CHOPTANK RIVER)	S	3-7	1.6	6.1	M	-50000 CU.YDS/YR ( )	TALBOT
#96 CASTLE HAVEN POINT– CHAPEL CREEK (CHOPTANK RIVER)	S	2	1.6	6.1	M	30,000 CU.YDS/YR (-)	DOR-CHESTER
#97 TODDS POINT	H	5	1.5	6.1	M	40,000 CU.YDS/YR ( )	DOR.
#98 COOK POINT– COVEY POINT	L	5-9	1.5	6.0	H	-65,000 CU.YDS/YR. (+,-)	DOR.
#99 MILLS POINT– HILLS POINT	H	2-7	1.5	5.2	H	20,000 CU.YDS/YR. (+)	DOR.

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#100 HILLS POINT – RAGGED POINT	M	1-7	1.4	5.1	M	-30,000 CU.YDS/YR (+)	DOR
#101 JAMES ISLAND	H	1-7	1.3	5.1	M	-70,000 CU.YDS/YR (–)	DOR
#102 HOOPER POINT – OYSTER COVE	H	1-7	1.3	5.1	M	35,000 CU.YDS/YR ( )	DOR
#103 OYSTER COVE – PUNCH ISLAND CREEK	H	2-5-8	1.3	5.1	M	+10,000 CU.YDS./YR (+,-)	DOR
#104 PUNCH ISLAND CREEK – BARREN ISLAND GAP	H	2-9	1.5	5.2	M	+5,000 CU.YDS/YR (+)	DOR
#105 BARREN ISLAND	H	2-8	1.5	5.6	M	-18,000 CU.YDS/YR (–)	DOR
#106 HOOPER ISLAND	M	3-9	1.7	5.6	M	+25,000 CU.YDS/YR ( )	DOR
#107 HOOPER ISLAND SOUTH END	L	2-9	1.8	5.6	H	-50,000 CU.YDS/YR ( )	DOR
#108 CRAB POINT – NORMAN COVE	—	—	1.8	5.7	H	+25,000 CU.YDS/YR ( )	DOR

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#109 NORMAN COVE – HOPKINS COVE	—	—	1.7	5.7	H	0 ( )	DOR
#110 BLOODSWORTH ISLAND	—	1	1.8	5.7	H	0 ( )	DOR
#111 FISHING BAY – SANDY ISLAND	L	1-9	1.9	5.8	M	+25000 CU.YDS/YR (+)	DOR
#112 CLAY ISLAND – SANDY ISLAND	S	1-9	2.0	5.8	M	-20000 CU.YDS/YR ( )	DOR
#113 ROARING POINT – NANTICOKE POINT	M	2-3-8	2.1	5.8	M	15,000 CU.YDS/YR ( )	WICOMICO
#114 LONG POINT – HAINES POINT	L	3-8	2.0	5.7	M	30,000 CU.YDS/YR ( )	SOMERSET
#115 HAINES POINT – TWIGGS POINT (DEAL ISLAND)	L	3-9	1.9	5.6	M	-6000 CU.YDS/YR ( )	SOM.
#116 TWIGGS POINT – WEBSTER POINT (DEAL ISLAND)	L	3-9	1.9	5.5	M	2000 CU.YDS/YR ( )	SOM.
#117 WEBSTER POINT – CLAW POINT (LITTLE DEAL ISLAND)	M	1-9	1.9	5.4	M	15000 CU.YDS/YR	SOM.

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#118 SOUTH MARSH ISLAND	—	1-9	1.7	5.4	H	-40,000 CU YDS/YR ( )	SOM.
#119 TEAGUE CREEK-DRUM POINT (MANOKIN RIVER)	S	2-9	1.9	5.5	M	+25,000 CU YDS/YR ( )	SOM.
#120 DRUM POINT-PAT ISLAND	S	2-9	1.9	5.4	H	25,000 CU YDS/YR ( )	SOM.
#121 FLATCAP POINT-(JANES ISLAND)	S	2-9	1.9	5.3	M	30,000 CU YDS/YR ( )	SOM.
#122 JANES ISLAND	M	2-9	1.9	5.3	M	-20,000 CU YDS/YR ( )	SOM.
#123 CEDAR ISLAND	M	2-9	1.9	5.2	M	-15,000 CU YDS/YR ( )	SOM.
#124 CEDAR STRAIGHTS-APE HOLE CREEK (POCOMOC SOUND)	L	2-9	1.9	—	M	-30,000 CU YDS/YR ( )	SOM.
#125 APE HOLE CREEK-EAST CREEK (POCOMOC SOUND)	L	2-9	1.9	—	M	-35,000 CU YDS/YR ( )	SOM.
#126 SMITH ISLAND-TANGIER SOUND	S	2-9	1.6	4.9	M	-10,000 CU YDS/YR ( )	SOM.

TABLE 5.6 CHESAPEAKE BAY SHORELINE CATEGORIZATION

REACH NUMBER AND LOCATION	HISTORIC EROSION RATE	SHORE TYPE	MEAN TIDE RANGE (ft.)	"100-yr." STORM SURGE (ft.)	WAVE ENERGY	COMPUTER PREDICTION OF NET LITTORAL DRIFT RATE AND DIRECTION ( ) FROM AERIAL PHOTOGRAPHS	COUNTY
#127 SMITH ISLAND - FOG POINT - BIG THOROFARE	—	2.9	1.6	5.0	H	0 ( )	Som.
#128 SMITH ISLAND BIG THOROFARE - HOG NECK	H	2.9	1.7	5.1	H	-10,000 CU. YDS/YR ( )	Som.

CHAPTER VI

STATISTICAL MODELLING OF HISTORICAL SHORE EROSION PATTERNS

Randall K. Spoeri

A. Introduction

In the previous chapter, variations were described in several coastal processes, and a subjective, qualitative analysis was performed to examine the relationship between the historical rate of coastal retreat and each environmental variable: terrain, tide range, storm conditions, wave climate, and littoral drift. A general classification of shoreline characteristics was also compiled in Table 5.6 for 128 separate reaches of Maryland's shoreline on the Chesapeake Bay and on the lower Potomac River. In this chapter, a more objective statistical approach is used to analyze the information presented in Table 5.6, and to illustrate the statistical utility of the different environmental variables to estimate the erosion rate on a general Bay-wide basis.

For this portion of the study, six variables were subjected to further analysis:

- Historical Erosion Rate
- Dominant Shoreline Type
- Mean Tide Range
- "100-Year" Storm Surge
- Wave Energy
- Littoral Drift

These variables were defined in sections of the previous chapter. For those reaches having missing values for certain variables, estimated values were provided from the detailed maps furnished by COER, Inc. to the

Tidewater Administration. Furthermore, dominant shoreline type (Chapter V, Section D) contained a very large number of categories (20). For the sake of simplifying the analysis discussed in the following sections of this chapter, these detailed categories for shoreline terrain were synthesized into seven general divisions:

- Beach
- Beach with Headland less than 20 feet high
- Beach with Headland greater than 20 feet high
- Headland less than 20 feet high
- Headland greater than 20 feet high
- Shore
- Marsh

This determination was made for each reach, by relying on the detailed observations of the shoreline compiled by student interns from Anne Arundel Community College. A final correction which was applied to the information in Table 5.5 was to eliminate twenty-one reaches which were not considered to be necessarily representative of the main Chesapeake Bay shoreline in Maryland. These reaches were Numbers 1-13 (all along the lower Potomac River), Numbers 78-82 (back side of Kent Island and Eastern Bay) and Numbers 108-110 (lower Eastern Shore). The remaining one hundred seven reaches, or units of analysis, served as the data base for subsequent statistical analysis. Historical erosion rates on each of these reaches is predicted (or modelled) as a function of the remaining five variables listed on the previous page, by organizing the values from Table 5.5 (with changes noted above) into a computerized data file, and performing various analyses.

The statistical analyses performed for this data can be organized into three categories:

- Descriptive Statistical Analysis
- Regression Analysis
- Discriminant Analysis

For these analyses, three statistical computer program package sources were employed:

- Statistical Package for the Social Sciences (SPSS) - Nie et. al., (1975).
- Minitab- Ryan, et. al., (1981)
- Miscellaneous Special Purpose Programs

B. Descriptive Statistical Analysis

Descriptive statistical methods are useful in summarizing large amounts of data as well as for examining the characteristics, distributional properties, and interrelationships for variables under analysis. In this study, a variety of summary statistics and graphical methods were used to carefully examine the data in an effort to reveal patterns, extremely large or small values, and, in general, simply to explore the data structures. Such summary measures as the mean, standard deviation, and correlation coefficient were calculated and examined for the variables described previously. In addition, a variety of graphical displays were produced and evaluated. These included the more traditional histograms, bar charts, and scatter plots, as well as some of the newer methods generally referred to as "Exploratory Data Analysis" (EDA) methods. A full description of the EDA methods is given by McNeil (1977). Such EDA displays as "Stem-and-Leaf", "Boxplots", and "Smooths" were used.

The results of these descriptive analyses suggested a number of relationships and patterns which may be useful in subsequent studies, and also helped to explain some results observed in the regression and discriminant analyses. In general however, no "bad" data were identified although several seemingly nonlinear relationships were apparent. This fact will be noted in later discussion.

C. Regression Analysis

For this study, it was desirable to describe the joint relationship between a single dependent variable Y, the historic erosion rate on any particular reach, and several independent variables X_i, the values for wave climate, tide range, storm conditions, littoral drift rates, and shoreline terrain listed in Table 5.5. As mentioned in Chapter V, the historic erosion rate classifications represent only the gross characteristics within each shoreline reach, and the classification is necessarily subjective at times when the shoreline variations are irregular within the reach. This needs to be kept in mind when considering the results presented below.

In order to determine if historic erosion rates could be predicted confidently on the one hundred-seven reaches as a function of the five explanatory variables, a multiple linear regression model was hypothesized:

$$Y_i = \beta_0 + \beta_1 X_{1i} + \beta_2 X_{2i} + \beta_3 X_{3i} + \beta_4 X_{4i} + \beta_5 X_{5i} + \varepsilon_i \qquad (6.1)$$

where:

i indexes the 107 reaches

Y_i = historic erosion rate of the ith reach

X_{1i} = dominant shore type of the ith reach

X_{2i} = mean tide for the ith reach

X_{3i} = "100-year" storm surge for the ith reach

X_{4i} = wave energy of the ith reach

X_{5i} = littoral drift of the ith reach.

The β_i values indicate model parameters to be estimated from the data and ε_i represents a random error term, indicating the "lack of fit" of the explanatory variables to Y_i. A thorough treatment of regression analysis is provided in Draper and Smith (1968).

Now, in order to properly use a multiple regression analysis, each of the variables must be numerical. Since Y, X_1, and X_4 were categorical, an additional step was necessary prior to performing the regression analysis. This involved the creation of "indicator variables" to represent the categories defined by variables X_1 and X_4. Y was quantified by letting

$$Y = \begin{cases} 0, & \text{if historic erosion rate was A (accretion)} \\ 1, & \text{" " " " " S} \\ 2, & \text{" " " " " L} \\ 3, & \text{" " " " " M} \\ 4, & \text{" " " " " H} \end{cases}$$

Then for X_{1i}, six indicator or "dummy" variables were introduced, where

$$D_{1i} = \begin{cases} 1, & \text{if the ith reach was a beach} \\ 0, & \text{if otherwise} \end{cases}$$

$$D_{2i} = \begin{cases} 1, & \text{if the ith reach was a beach with headland less than 20 feet} \\ 0, & \text{if otherwise} \end{cases}$$

$$D_{3i} = \begin{cases} 1, & \text{if the ith reach was a beach with headland greater than 20 feet} \\ 0, & \text{if otherwise} \end{cases}$$

$D_{4i} \equiv \begin{cases} 1, & \text{if the ith reach was a headland less than 20 feet} \\ 0, & \text{if otherwise} \end{cases}$

$D_{5i} \equiv \begin{cases} 1, & \text{if the ith reach was a headland greater than 20 feet} \\ 0, & \text{if otherwise} \end{cases}$

$D_{6i} \equiv \begin{cases} 1, & \text{if the ith reach was a shore} \\ 0, & \text{if otherwise} \end{cases}$

Obviously, if $D_{1i} = D_{2i} = .. = D_{6i} = 0$, then the reach is a marsh.

The variable X_4 in equation 6.1 was similarly transformed into two indicator variables for the regression analysis. A thorough treatment of the use of indicator variables is given by Neter and Wasserman (1974).

A computer program was then written to transform the previously structured data file into a completely numerical data file incorporating the transformed Y, the six indicator variables representing X_1 and the two indicator variables for X_4.

Finally, the multiple linear regression model fit to the erosion data file can be expressed as (omitting the reach subscript "i" for simplicity):

$$Y = \beta_0 + \beta_{11}D_1 + \beta_{12}D_2 + \beta_{13}D_3 + \beta_{14}D_4 + \beta_{15}D_5 + \beta_{16}D_6 + \beta_2X_2 + \beta_3X_3 + \beta_{41}D_7 + \beta_{42}D_8 + \beta_5X_5 + \varepsilon \quad (6.2)$$

Hence, the final model involved the use of 11 explanatory variables, for which 12 parameters were to be estimated. The resultant estimates "b_i" and their standard errors (A measure of an estimate's "stability"), are shown in Table 6.1. At the $\alpha = 0.05$ significance level, only variables D_3 (dominant shore type = beach with headland greater than 20 feet high) and X_3 ("100-year" storm surge) had coefficients which were statistically significantly different from zero. Furthermore, the "R^2" statistic was 0.3077 for this model. This statistic represents the amount of variation

Table 6.1

Parameter Estimates and Standard Errors for the Linear Regression Model

Variable	Regression Coefficient (bi)	Standard Error (S(bi))
intercept	3.402917	
D_1	0.228297	0.266250
D_2	-0.029221	0.375726
D_3	-1.008549	0.422220
D_4	-0.702722	0.375946
D_5	-0.977971	0.638387
D_6	-0.077129	0.303585
X_2	-0.229369	0.396837
X_3	-0.153162	0.049054
D_7	-0.498455	0.485153
D_8	-0.204231	0.221474
X_5	0.040058	0.031133

in historic erosion rates which was explained by or predicted using the eleven independent variables. The largest which R^2 can be is 1.0 if there is perfect fit of all the data to the predictive equation. Accordingly, only 30.77% of the variation in Y was predicted using the linear model (equation 6.2).. Consequently, this particular model does not provide an adequate tool for suitably predicting, or modelling, the historical pattern of erosion on a general Bay-wide basis for Maryland.

As a further check, a stepwise regression analysis was performed to determine which of the 11 predictor candidates would be most useful. This analysis identified only two variables: variable X_3 ("100-year" storm surge), and variable D_1 (dominant shore type = beach) as being the only useful variables at the 0.05 level of significance. R^2 for this two variable model was only 20.73% explained variation.

In summary, the results of the regression analysis indicate that a multiple linear regression model does not provide a useful tool to suitably predict erosion rate as a function of environmental variables for portions of the main Chesapeake By shoreline in Maryland. It is important to note, however, that various technical assumptions may well have been violated in performing the regression analysis described in this section. These assumptions include linearity of the data, independence of the observations, and common variance of the Y's across the range of the X_i's. Based on the descriptive analysis reported in section B of this chapter, and on a post-regression residual analysis, it is suspected that, at a minimum, nonlinearity exists in the data. This means that the statistical relationship between the pattern of historic erosion around the northern Bay shoreline and the environmental variables might be more accurately assessed by introducing polynomial, multiplicative, or possibly exponential functions of the variables into the regression model. However the physical meaning of these results, should they prove to be a closer predictive fit of the data, would have to be carefully interpreted.

Other assumptions may also have been violated. Nevertheless, it was not considered within the scope or time frame of the present study to investigate all potential statistical relationships in the data, or to seek remedial measures or alternative models. That is a subject for later study.

D. Discriminant Analysis

In an effort to confirm the conclusions reached based on the regression analysis, an alternative statistical method was applied to the erosion data file. This method is called discriminant analysis. A good overview of the technique is given by Kendall (1975), while a thorough treatment of the subject can be found in Lachenbruch (1975).

The basic objective of a discriminant analysis is to decide, on the basis of measured variables, to which of two or more predefined groups a particular unit of analysis should be assigned or classified. In this case, the unit of analysis was a shoreline reach along the main Chesapeake Bay shoreline in Maryland, and the variables were those described previously for use in the regression analysis. There were five predefined groups:

(1) Reaches experiencing accretion (A)

(2) Reaches experiencing a slight historical erosion rate (S)

(3) Reaches experiencing a low historical erosion rate (L)

(4) Reaches experiencing a moderate historical erosion rate (M)

(5) Reaches experiencing a high historical erosion rate (H).

When performing discriminant analysis, one begins with a set of units which have been classified and for which measured variables are available. The technique then involves using this information to create a classification scheme (statistical model) by which future units, whose group affiliation is not known, can be classified. In this process, it is frequently of interest to also determine which of the measured variables is most useful or "important" in distinguishing among groups, although this was not the purpose of this study. Here, the goal was to be able to develop a mathematical expression (model) by which "future" reaches could be categorized into one of the above groups. In this analysis, the SPSS (see Nie, et. al., (1975)) discriminant analysis program was used. A variety of information is provided by this computer program. A portion of this information is given in Table 6.2. The principal result of interest was the "misclassification rate." This indicates the proportion of reaches which would be incorrectly classified if they were to be "reclassified" using the discriminant analysis methodology previously developed. The percentage of all "classified" reaches correctly "reclassified" was 47.66%, so that the overall "chances" of properly classifying a shoreline reach on the basis of

Table 6.2
Prediction Results Based on
Discriminant Analysis

Actual Historical Erosion Rate	Number of Reaches	Predicted Historical Erosion Rate A	S	L	M	H
A	1	1 100%	0 0%	0 0%	0 0%	0 0%
S	39	2 5.1%	16 41.0%	12 30.8%	5 12.8%	4 10.3%
L	38	3 7.9%	4 10.5%	19 50.0%	4 10.5%	8 21.1%
M	21	1 4.8%	0 0%	4 19.0%	9 42.9%	7 33.3%
H	8 107	0 0%	0 0%	0 0%	2 25.0%	6 75.0%

Overall number correctly classified = 51 out of 107
Overall % correctly classified = 47.66%
Overall % incorrectly classified = 52.34%

the measured variables is less than 50%. Therefore, this approach seems to confirm the conclusions reached from the regression analysis. As with the regression analysis, no attempt was made to examine adherence to the technical assumptions required by discriminant analysis. This would be an important next step in the modelling effort.

E. Summary

In conclusion, the initial results seem to indicate that modelling the pattern of historic erosion rates around the edges of the main Chesapeake Bay in Maryland cannot be suitably done by using traditional regression or discriminant analysis procedures. This could be due to several causes. Among them are:

- Nonlinearities in the data
- Violations of other technical assumptions
- Poor data quality
- Relevant variables not included.

The analysis presented in this chapter is necessarily preliminary in nature, due to lack of useful results from only standard statistical approaches. To achieve the goal of explaining in a mathematical manner the reasons why different shoreline reaches on the Chesapeake Bay in Maryland experience varying rates of historic coastal retreat, further statistical analysis and research would be necessary. This research would include the use of nonlinear models, various other data transformations, and alternative statistical methods, such as time series analysis. Even if these procedures were performed and were found to yield useful and encouraging results from a statistical point of view, the physical meaning and usefulness of the results to Bay managers would have to be carefully assessed.

CHAPTER VII

LAND USE AND SHORE EROSION

Vic Klemas, Hsiang Wang

Robert Biggs, Robert Dalrymple

A. Introduction

Land use is one factor which has been often cited as influencing erosion of shoreline (Dolan, et al. 1980; Pilkey, et al. 1978), but the evidence is less than conclusive. A portion of this study was allocated to an examination of land-use changes in four selected shorefront areas (Figure 7.1) for evidence of impact on the shore erosion rate.

B. Methods

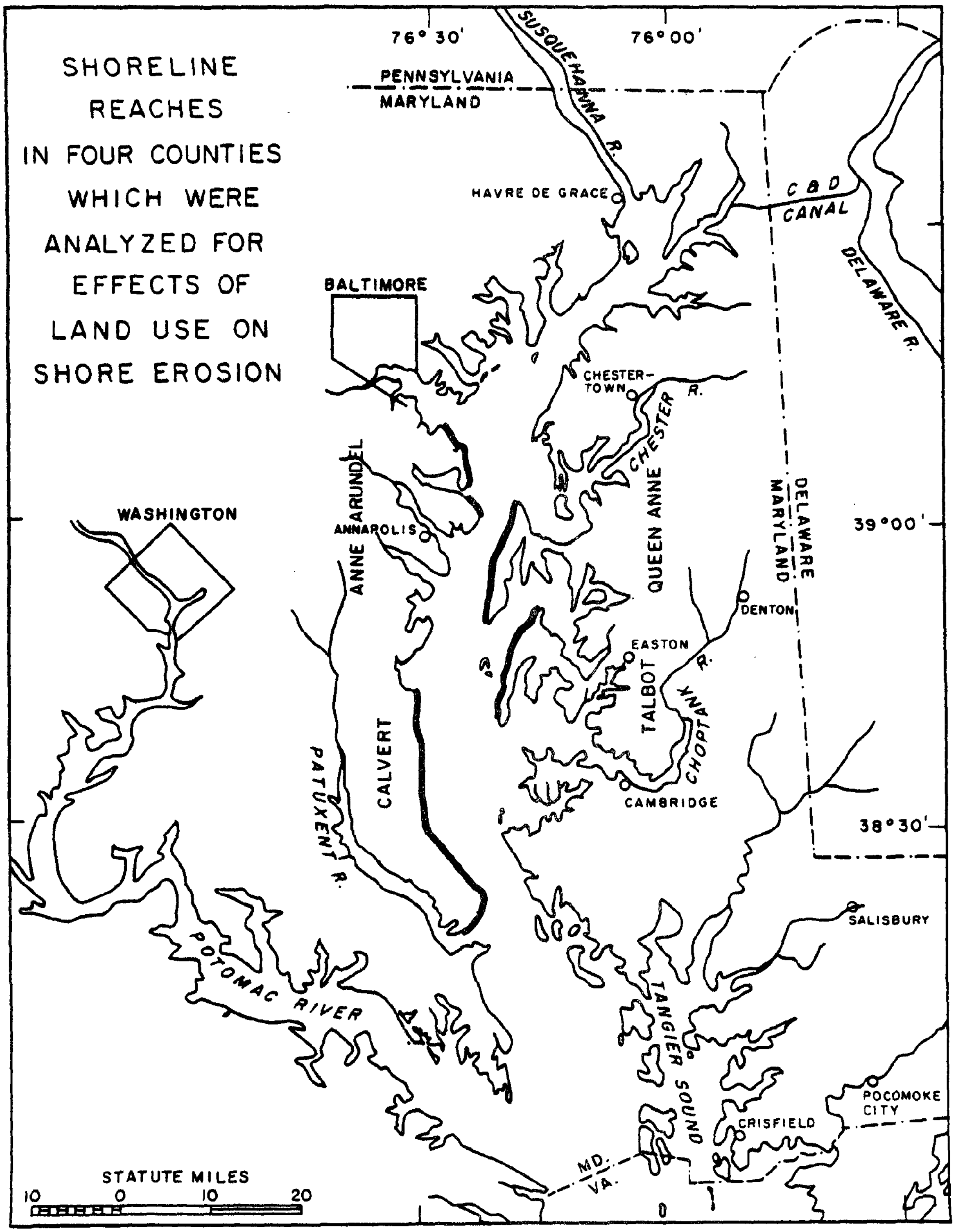
The land-use analysis was performed directly on aerial photo index sheets which are available from the Agricultural Stabilization and Conservation Service of the U.S. Department of Agriculture. Aerial photo mosaic index sheets at a scale of 1:40,000 were chosen, since the individual photographs at a scale of 1:20,000 would have been much too costly to obtain and too time-consuming to analyze. LANDSAT satellite imagers is also available but does not have the required spatial resolution and is available only from 1972 to the present time.

The aerial photo index sheets were flown as follows:

1.	Queen Annes County	1937,	'52,	'57,	'64,	'72
2.	Talbot County	1937,	'52,	'57,	'64,	'72
3.	Anne Arundel County	1938,	'52,	'57,	'63,	'70
4.	Calvert County	1938,	'52,	'57,	'64,	'71

Opposite: Figure 7.1. Shoreline reaches in four counties which were analyzed for land-use relationships to shore erosion between 1938-1971.

Figure 7.1


SHORELINE REACHES IN FOUR COUNTIES WHICH WERE ANALYZED FOR EFFECTS OF LAND USE ON SHORE EROSION
76° 30'
76° 00'
PENNSYLVANIA
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C & D CANAL
DELAWARE R.
BALTIMORE
CHESTER-TOWN
CHESTER R.
ANNE ARUNDEL
ANNAPOLIS
QUEEN ANNE
DELAWARE
MARYLAND
39° 00'
WASHINGTON
DENTON
EASTON
TALBOT
CHOPTANK R.
CALVERT
PATUXENT R.
CAMBRIDGE
38° 30'
SALISBURY
POTOMAC RIVER
TANGIER SOUND
POCOMOKE CITY
CRISFIELD
MD.
VA.
STATUTE MILES
10 0 10 20

Land-use was mapped on these sheets by visual photo-interpretation for a 250 meter wide strip along the shoreline in each year. Table 7.1 summarizes the patterns of development in the four test areas. The land-use categories which were identified included the following:

- Lightly Developed (L) (One house per acre or lower density)
- Heavily Developed (H) (One house per 1/2 acre or higher density; marinas; commercial development; etc.)
- Agricultural Land (A) (Cultivated or uncultivated fields reaching shoreline)
- Agriculture with Protective Strip (S) (Fields separated from beach by protective strip of trees or shrubs)
- Forest and Uplands Vegetation (U) (Forest, shrubs, and other uplands vegetation)
- Salt or Brackish Estuarine Marsh (M) (Includes river marshes in Calvert County)

The interpreted shoreline lengths may be smaller than the actual shoreline distances on a map because creek mouths, small bays, inlets and other curved shoreline stretches were approximated by short straight segments. The accuracy of the land-use interpretation for the 1952, 1963-64 and 1970-72 dates is estimated at about $\pm$ 7.5% of the stated lengths. For the 1937-38 time period, it is expected that the relative errors on the land-use maps are less than $\pm$ 15%. Since land-use changes between 1937-38 and

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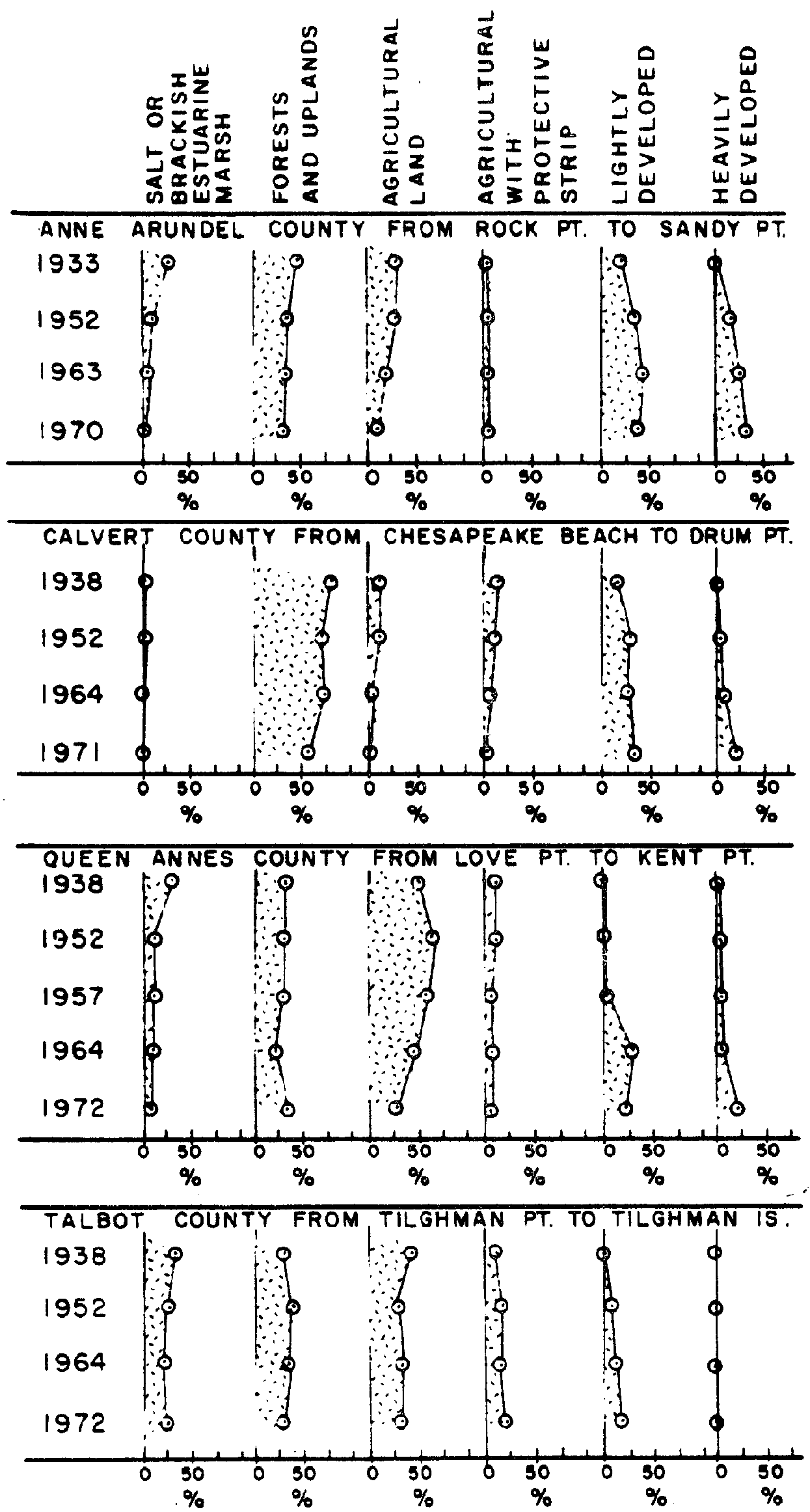
Opposite: Table 7.1. Land-use data and erosion rates.

Table 7.1



SHOREFRONT LAND USE CHANGES

1952 were small when compared to later periods (Table 7.1), the expected overall accuracy of the percentage land-use change results is estimated to be within about ± 10%.

C. Results

As expected, agricultural land-use has decreased and developed areas have increased along the shoreline in all four test areas. This is particularly evident at the eastern shore sites. The development of the western shore was much more rapid than that of the eastern shore sites. The marsh areas have also decreased with time at each study site. The loss of forested or uplands areas to development appears slow, primarily because in the analysis, a large forested lot (e.g. ten acres) with a house on it was classified as forest or uplands (U) and not lightly developed (L).

Because the land-use patterns are constantly changing, it is not possible to relate this factor to the trends in shore erosion which must be measured over time-scales of decades. It is not possible to identify reaches greater than 0.5 kilometers in the four test areas which have had a single land-use classification since 1937, and reaches of sufficient length and stable land-use elsewhere along the Chesapeake Bay are probably very rare. The selection of 0.5 kilometers as a minimum length for study is arbitrary, but this is the smallest reach length which is regarded as suitable for analyzing variations in historical rates of shoreline erosion.

In summary, the attempt to relate land-use characteristics to rates of coastal retreat was not successful because it was not possible to identify reaches which have had a similar land-use classification for a sufficient period of time. But this does not mean land-use cannot be significant in shore erosion. The DNR Major Facilities Siting Study (1977) "Environmental

Assessment Handbook" contains the following passage on page 55-56 concerning erosion effects due to land use:

> "Shoreline erosion is aggravated by new development occurring where the rate of erosion is high. Shoreline development activities may weaken the structure of the bank, causing collapse and slumping. Alteration of runoff and groundwater flow over or through the bank increases its susceptibility to erosion. Impervious surfaces such as paved roads, parking lots, structures, and agricultural drainfields may adversely affect the structure of the bank and the surface runoff and groundwater flow processes. Removal of vegetation on banks also increases the shoreline's susceptibility to erosion. With the loss of the knotting and binding effect of roots, banks are directly exposed to the adverse effects of erosion."

This is an accurate appraisal of the situation, but there is simply no suitable data collected so far to relate these effects to differences in shore erosion over sufficiently long periods of time.

A priority for future investigations would be the classification of all benchmark erosion stations maintained by state, local and federal agencies as to historical land-use, geology, erosion rate and littoral drift. Within 20-30 years, this might provide a suitable data base for analysis of the effect of land use on shore erosion.

Chapter VIII
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APPENDIX A

Shoreline Sediments Along the Chesapeake Bay in Maryland

Robert Biggs, Robert Dean,

Hsiang Wang and Robert Dalrymple

The table on the next page describes the nature of the geologic formations which are found along the Chesapeake Bay shoreline in Maryland. These sediments are part of the Atlantic Coastal Plain and are as old as the early Cretaceous Period (approx. 70 million years before present). The formations are largely horizontal sedimentary beds of sand, silt, and clay. Recent alluvial and marsh deposits also occur in certain environments. The formations are essentially horizontal in outcrop and intersect the shoreline in a variety of terrains which range from the high cliffs of Calvert County to the marshy lowlands of the southeast.

The major source of information for many of the geologic descriptions in the table is the Geologic Map of Anne Arundel County (Glaser, 1976), and the Geologic Map of Maryland (Cleaves, et al., 1968). Modifications and additions are from Geography and Geology of Maryland (Vokes, 1957), and Glaser's (1960) study of the Magothy and Potomac Gp. sediments, the most recent intensive study of any of the formations in the county. The most recent mapping of the county was done by Glaser (1976) using a standard scale of 1:62,500. Overall, this map can be considered very accurate and the modern standard for description of these sediments.

The major source of information for the description of the Talbot Formation is the Geologic Map of Baltimore County and City (Crowley, et al., 1976). Modifications are from Vokes (1957). Mapping is of standard scale of 1:62,500.

The source of description of the combined Potomac Gp. sediments is the Geologic Map of Harford County (Owens, 1963) in standard 1:62,500 scale. Sources of geologic description for the lower western shore include the Geologic Map of St. Marys County (Clark, 1902), scale 1:62,500, the Geologic Map of Calvert County (Clark, 1902); and Environmental History of Maryland Miocene (Gernant, et al., 1971).

Sources of geologic information on Kent, Queen Annes, and Talbot County are the maps of the three counties (Clark, 1915, 1916). These are all standard 1:62,500 scale.

Owens and Denny (1979) have recently completed a new interpretation of the stratigraphy in some areas of the Delmarva Peninsula and have reclassified those sediments. The descriptions used in the table reflect their work. This involves renaming the Talbot, Pamlico and Princess Anne lowland deposits as the Kent Island Formation. Other sources of information on the geology of the lower eastern shore include the Geologic Map of Wicomico County (Owens and Denny, 1979) and a map, in U.S.G.S. Professional Paper #1067-A, of scale 1:1,267,200 (Owens and Denny, 1979).

Next Pages: Table A.1. Shoreline sediments along the Chesapeake Bay in Maryland.

Table A.1

Shoreline Sediments Along the Chesapeake Bay in Maryland

Artificial Fill - Sand, gravel and clay. Construction debris and dredge spoil also common. In most counties this material is used as nourishment at beach sites and inlets and as foundation in nearshore construcion projects and landfills. Extensive areas of the City of Baltimore are comprised of this material.

Tidal Marsh - Silty clay to fine sand with woody debris and organic matter abundant. Most abundant in Dorchester, Wicomico, and Somerset counties.

Alluvium - Interbedded sand, silt-clay and gravel. Beach deposits are well sorted, fine-to-medium grained sands. Marsh deposits are dark, organic-rich mud. Present in all counties except Dorchester, Wicomico, and Somerset. In Baltimore County, the natural distribution of alluvium has been heavily modified by artificial fill operations.

Talbot Fm. - Interbedded muddy sand, silt, and clay; lower portions are more typically pebbly sand or gravel. In all counties except Cecil, Dorchester, Wicomico and Somerset. This formation typically underlies low flat areas bordering the Bay and shores of the larger estuaries.

Parsonburg Sand Fm. - Mostly moderately sorted, medium-to-coarse grained loose, yellow sand. Found only sparingly in the coastal areas of Wicomico County.

Kent Island Fm. - Sand interstratified with thin beds of dark gray silt or silty fine-grained sand. Gravelly sands common at base. Found along the shoreline in portions of Dorchester, Wicomico, and Somerset Counties.

Lowland Deposits - Gravel, sand, silt, and clay, with cobbles and boulders near the base. Also contains reworked glauconitic sands. Found principally in Cecil County.

Terrace Deposits - Medium-to-coarse grained pebbly sand, with subordinate mud. Found in minor amounts along the shoreline in Anne Arundel County.

Wicomico Fm. - Loam, clay, sand, gravel and boulders. Found between 90 and 200 feet elevation along the shoreline in Calvert and St. Mary's Counties.

Upland Deposits - Typically cross-bedded, poorly-sorted, medium-to-coarse grained sand and gravel, with boulders near base and subordinate silts and clays. Found in Cecil County.

Table A.1

Shoreline Sediments Along the Chesapeake Bay in Maryland

St. Mary's Fm. - Bluish clay, sandy clay and marl. Sand tends to be fine-grained. Found along Calvert and St. Mary's County shorelines.

Choptank Fm. - Yellow sandy clay and marl. Found along Calvert and St. Mary's County shorelines.

Calvert Fm. - Fine-grained sand, silt and diatomaceous silt. Basal beds (Fairhaven Member) contain much poorly-sorted medium sand overlain by highly diatomaceous silt. Found along Anne Arundel, Calvert, St. Mary's, and Queen Anne's County shorelines.

Nanjemoy Fm. - Fine-to-medium grained, poorly-sorted clayey sand with subordinate silt and silty clay. Found along Anne Arundel County shoreline.

Aquia Fm. - Well-sorted, medium-grained, clean-to-moderately clayey, glauconitic sand. Cemented in places, but typically soft and friable. Found along Anne Arundel County shoreline.

Monmouth Fm. - Fine-grained, variably glauconitic sand and micaceous, clayey silt. Found along Anne Arundel & Kent County shorelines.

Matawan Fm. - Dark gray, micaceous, and glauconitic, fine-grained sand and silt. Found along Cecil and Kent County shorelines.

Magothy Fm. - Fine-to-coarse grained sand interstratified with silt-clay and subordinate pebbly sand or gravel. Found along Anne Arundel and Cecil County shorelines.

Potomac Gp - Found along Anne Arundel, Baltimore, Harford, and Cecil County shorelines.

Sand-Gravel Facies - Interbedded quartz sand, pebbly sand, gravel and subordinate mud.

Silt-Clay Facies - Clay, silt and subordinate fine-to-medium grained, muddy sand. Generally massive, compact and "tough" in nature.

Raritan Fm. - Interbedded sand, sandy clay, and clay. The sands are at times indurated. Found along Kent County shoreline.

Appendix B

Examples of New Atlas Maps

Several new sets of atlas maps were developed by COER Inc. as part of this study to aid in the planning for future shore erosion assessment and the siting of new erosion-control structures. The original copies of each atlas are are on file in the Coastal Resources Division of the Maryland Department of Natural Resources. The atlas maps include:

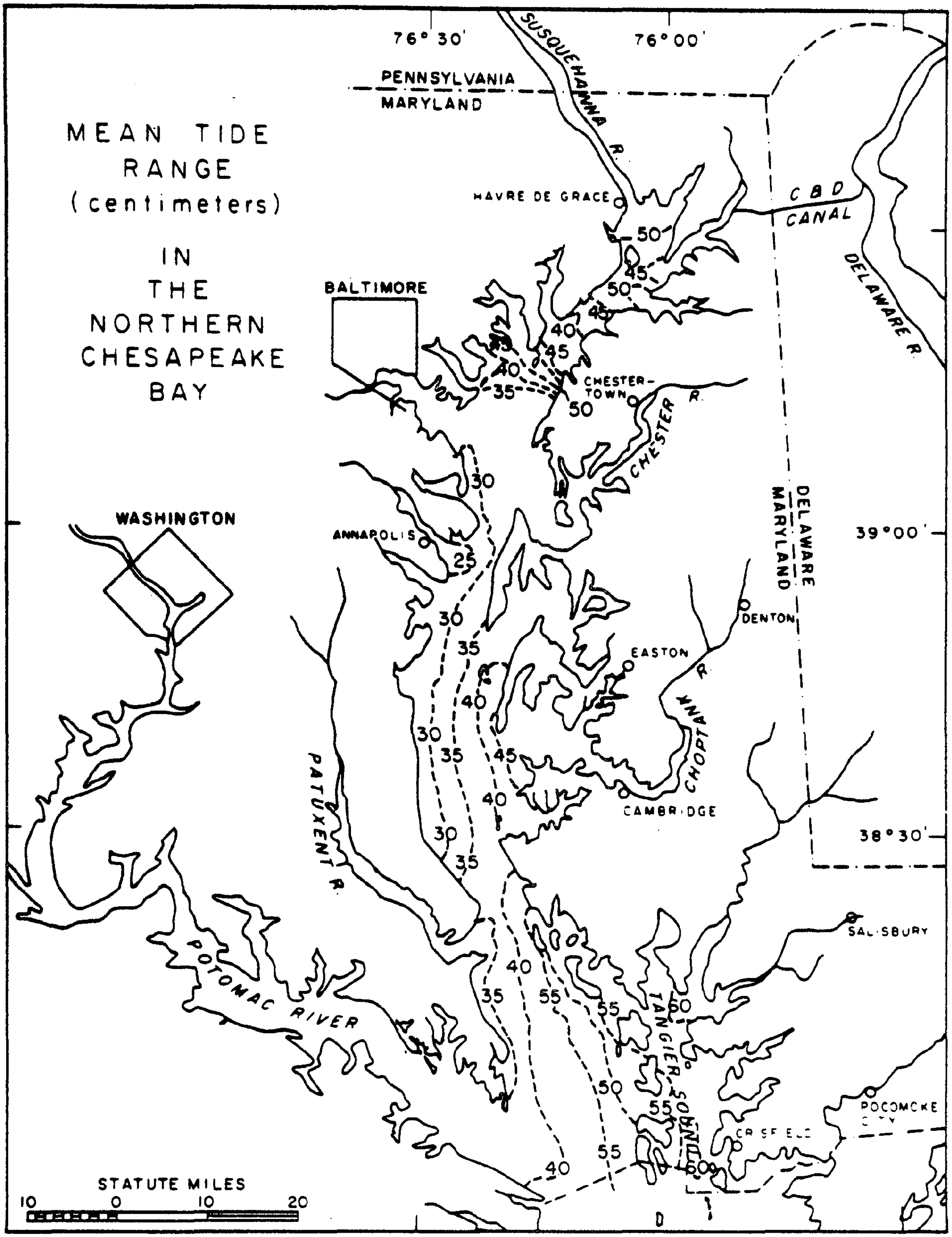
- a shore zone classification, discussed in Chapter V (pages 5-7 through 5-9). The 1972 aerial photos (1:12,000 scale) on file in the Wetlands Section of the Maryland DNR were used to classify the shoreline according to terrain. The classification scheme used was previously developed by Ahnert (1974) and is listed on page 5-7. The complete classification of the Maryland Chesapeake Bay shoreline is contained in a new atlas which consists of transparent copies of all 7 1/2 minute topographic quadrangle sheets of the Maryland portion of the Bay. An example of the new atlas product is shown in Figure 5.3 (Chapter V).

- a set of maps showing tidal elevations inside the Bay (mean tide range and Class 4 tide range). The method for computing the tide ranges was discussed in Chapter V (pages 5-12 through 5-16). The computer predictions of tide conditions were compiled onto a set of transparent Bay navigation charts (scale 1:80,000). The computer predictions were produced in metric units, and a simplified version of the charts is shown in Figure B.1 and Figure B.2.

- a set of maps showing Class 4 tidal currents and "100-year" storm surge heights predicted from a computer model. The method for predicting the storm surge heights was discussed in Chapter V (pages 5-17 through 5-23). The results of the computer predictions were compiled onto a transparent set of Bay navigation charts (scale 1:80,000). The computer predictions were produced in metric units, and a simplified version of the charts is shown in Figure B.3.

  For the prediction of maximum Class 4 tidal currents, the maximum tidal currents associated with each class of tidal range were estimated using the same numerical storm surge model developed by Chen (1978) as for "100-year" storm surge predictions. The model was calibrated by comparing the calculations with results by Hicks (1964) and the U.S. Tide Tables and U.S. Tidal Current Tables. The hydrodynamic model was used to generate tidal currents at 4-minute intervals for a complete tidal cycle using Class 4 tide as input (see Table 5.2). The maximum values were then determined. The results are printed on a transparent set of Bay navigation charts (scale 1:80,000).

Figure B.1



MEAN TIDE RANGE (centimeters) IN THE NORTHERN CHESAPEAKE BAY
76° 30'
76° 00'
39° 00'
38° 30'
PENNSYLVANIA
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C&D CANAL
DELAWARE R.
BALTIMORE
CHESTERTOWN
CHESTER R.
WASHINGTON
ANNAPOLIS
DELAWARE
MARYLAND
DENTON
EASTON
CHOPTANK R.
CAMBRIDGE
PATUXENT R.
POTOMAC RIVER
SALISBURY
TANGIER SOUND
POCOMOKE CITY
STATUTE MILES
10 0 10 20

Figure B.2

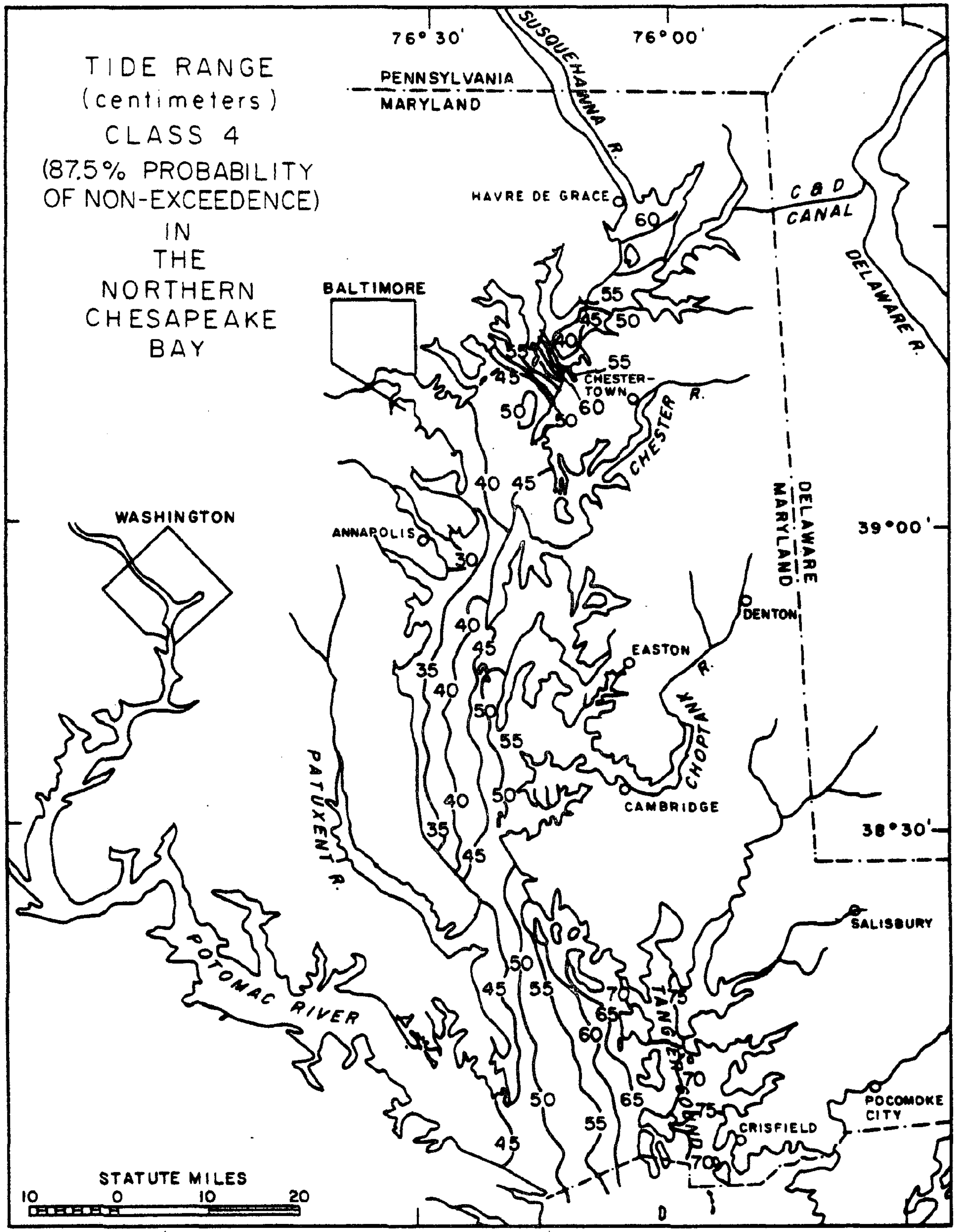
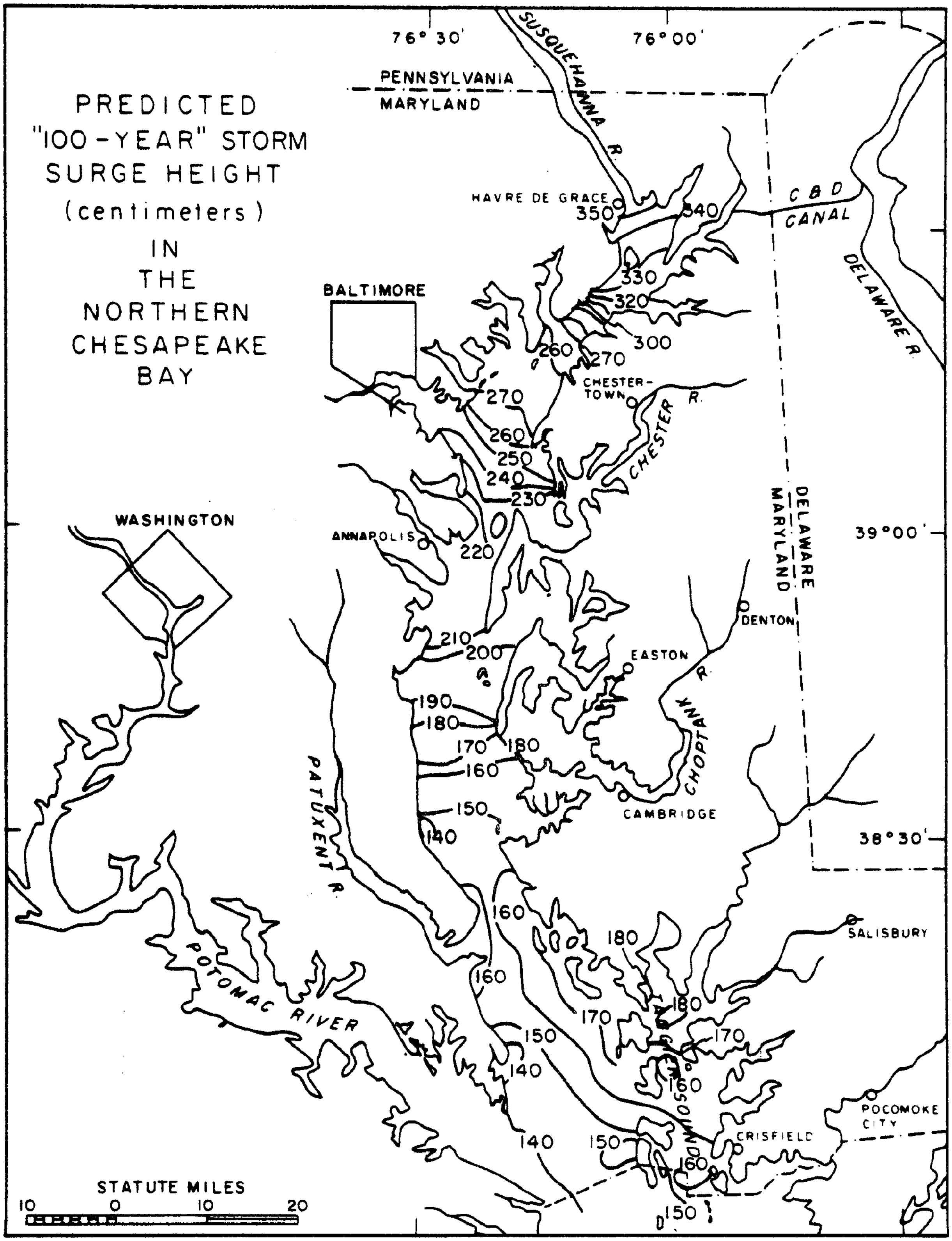

TIDE RANGE
(centimeters)
CLASS 4
(87.5% PROBABILITY
OF NON-EXCEEDENCE)
IN
THE
NORTHERN
CHESAPEAKE
BAY
76° 30'
76° 00'
39° 00'
38° 30'
PENNSYLVANIA
MARYLAND
DELAWARE
MARYLAND
SUSQUEHANNA R.
HAVRE DE GRACE
C & D
CANAL
DELAWARE R.
BALTIMORE
CHESTER-
TOWN
CHESTER R.
WASHINGTON
ANNAPOLIS
DENTON
EASTON
CHOPTANK R.
CAMBRIDGE
PATUXENT R.
POTOMAC RIVER
SALISBURY
TANGIER SOUND
CRISFIELD
POCOMOKE
CITY
STATUTE MILES
10 0 10 20

Figure B.3


PREDICTED
"100-YEAR" STORM
SURGE HEIGHT
(centimeters)
IN
THE
NORTHERN
CHESAPEAKE
BAY
76° 30'
76° 00'
39° 00'
38° 30'
PENNSYLVANIA
MARYLAND
DELAWARE
MARYLAND
SUSQUEHANNA R.
C & D CANAL
DELAWARE R.
HAVRE DE GRACE
BALTIMORE
WASHINGTON
ANNAPOLIS
CHESTER-TOWN
CHESTER R.
EASTON
DENTON
CHOPTANK R.
CAMBRIDGE
PATUXENT R.
POTOMAC RIVER
SALISBURY
POCOMOKE CITY
CRISFIELD
STATUTE MILES
10 0 10 20

- a set of maps showing wave conditions near the Maryland Bay shoreline. To establish wave statistics for erosion assessment, a wave hindcast model was used based on the shallow water wind-wave generation technique developed by Wilson (1965). It was further modified to account for the limited fetch width in the Bay. The procedures are discussed in Chapter V (pages 5-24 through 5-35). The complete annual wave predictions are presented graphically in the form of wave roses on a transparent set of Bay navigation charts (scale 1:80,000) for each of the 128 reaches listed in Table 5.6 (Chapter V). The range of highest predicted waves for each reach on Maryland's Bay shoreline are shown in the map in Figure 5.12.

- a set of maps showing potential rates of longshore movement of sediments (littoral drift). The method for computing the littoral drift rates, and for producing littoral drift roses was discussed in Chapter V (pages 5-37 through 5-45). The results of the computer predictions were compiled onto transparent copies of all 7 1/2 minute topographic quadrangle sheets of the Maryland portion of the Bay. An example of the new atlas product is shown in Figure 5.3 (Chapter V). The range of predicted littoral drift rates for each reacn on Maryland's Bay shoreline are shown in the map in Figure 5.17, and in Table 5.6 (Chapter V).

To convert centimeters to feet, multiply by 0.0328. Example:

45 centimeters x 0.0328 = 1.48 feet

To convert feet to centimeters, multiply by 30.48. Example:

1 foot x 30.48 = 30.48 centimeters

Appendix C
Glossary of Terms

These definitions are taken from a list compiled by the U.S. Army Corps of Engineers Coastal Engineering Research Center (1973).

ALONGSHORE - Parallel to and near the shoreline; same as LONGSHORE.

BANK - (1) The rising ground bordering a lake, river, or sea; of a river or channel, designated as right or left as it would appear facing downstream. (2) In its secondary sense, a shallow area consisting of shifting forms of silt, sand, mud, and gravel, but in this case it is only used with a qualifying word such as "sandbank" or "gravelbank."

BACKSHORE - That zone of the shore or beach lying between the foreshore and the coastline and acted upon by waves only during severe storms, especially when combined with exceptionally high water. It comprises the BERM or BERMS.

BEACH - The zone of unconsolidated material that extends landward from the low water line to the place where there is marked change in material or physiographic form, or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of a beach - unless otherwise specified - is the mean low water line. A beach includes FORESHORE and BACKSHORE.

BEACH PROFILE - A side view of the zone along the shoreline that extends landward from the water's edge to the toe of a dune or bluff.

BERM - A nearly horizontal part of the beach or backshore formed by the deposit of material by wave action. Some beaches have no berms, others have one or several.

BLUFF - High, steep bank at the water's edge. In common usage, a bank composed primarily of soil. See CLIFF.

BREAKER ZONE - Area offshore where waves break.

BREAKWATER - A structure protecting a shore area, harbor, anchorage, or basin from waves.

BULKHEAD - A structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

CLAY - Extremely fine-grained soil with individual particles less than 0.00015 inch in diameter.

CLIFF - High steep bank at the water's edge. In common usage, a bank composed primarily of rock. See BLUFF.

COBBLES - Rounded stones with diameters ranging from 3 to 10 inches. Cobbles are intermediate between GRAVEL and BOULDERS.

COAST - A strip of land of indefinite width (may be several miles) that extends from the shoreline inland to the first major change in terrain features.

COASTAL PLAIN - The plain composed of horizontal or gently sloping strata of clastic materials fronting the coast, and generally representing a strip of sea bottom that has emerged from the sea in recent geologic time.

COASTLINE - (1) Technically, the line that forms the boundary between the COAST and the SHORE. (2) Commonly, the line that forms the boundary between the land and the water.

COVE - A small, sheltered recess in a coast, often inside a larger embayment.

CREST - Upper edge or limit of a shore protection structure.

CREST OF BERM - The seaward limit of a berm.

CROSS SECTION - A vertical section (profile) of the surface, the ground, and/or underlying material, which provides a side view of the structure or beach (see beach profile).

CURRENT - A flow of water.

CURRENT, EBB - The tidal current away from shore or down a tidal stream. Usually associated with the decrease in the height of the tide.

CURRENT, FLOOD - The tidal current toward shore or up a tidal stream. Usually associated with the increase in the height of the tide.

CURRENT, LITTORAL - Any current in the littoral zone caused primarily by wave action, e.g., longshore current, rip current.

CURRENT, LONGSHORE - The littoral current in the breaker zone moving essentially parallel to the shore, usually generated by waves breaking at an angle to the shoreline.

CURRENT, TIDAL - The alternating horizontal movement of water associated with the rise and fall of the tide caused by the astronomical tide-producing forces.

DATUM, PLANE - The horizontal plane to which soundings, ground elevations, or water surface elevations are referred.

DEEP WATER - Area where surface waves are not influenced by the bottom. Generally, a point where the depth is greater than one-half the surface wavelength.

DEEPWATER WAVE - Waves which develop in water of sufficient depth that are not influenced by the friction of the bottom.

DEPTH - The vertical distance from a specified tidal datum to the sea floor.

DIFFRACTION (of water waves) - The phenomenon by which energy is transmitted laterally along a wave crest, when a part of a train of waves is interrupted by a barrier, such as a breakwater, the effect of diffraction is manifested by propagation of waves into the sheltered region within the barrier's geometric shadow.

DIURNAL - Having a period or cycle of approximately one TIDAL DAY.

DOWNDRIFT - The direction of predominant movement of littoral materials.

DRIFT (noun) - (1) Sometimes used as a short form for LITTORAL DRIFT (2) The speed at which a current runs. (3) Also floating material deposited on a beach (driftwood).

DUNES - (1) Ridges or mounds of loose, wind-blown material, usually sand. (2) BED FORMS smaller than bars but larger than ripples that are out of phase with any water-surface gravity waves associated with them.

DURATION - In wave forecasting, the length of time the wind blows in nearly the same direction over the FETCH (generating area).

EBB CURRENT - The tidal current away from shore or down a tidal stream; usually associated with the decrease in the height of the tide.

EBB TIDE - The period of tide between high water and the succeeding low water; a falling tide.

EMBANKMENT - An artificial bank such as a mound or dike, generally built to hold back water or to carry a roadway.

EMBAYED - Formed into a bay or bays, as an embayed shore.

EMBAYMENT - An indentation in the shoreline forming an open bay.

EQUILIBRIUM - A state of balance or equality of opposing forces.

EROSION - The wearing away of land by the action of natural forces. On a beach, the carrying away of beach material by wave action, tidal currents, littoral currents, or by deflation.

ESCARPMENT - A more or less continuous line of cliffs or steep slopes facing in one general direction which are caused by erosion or faulting.

ESTUARY - (1) The part of a river that is affected by tides. (2) The region near a river mouth in which the fresh water of the river mixes with the salt water of the sea.

FEEDER BEACH - An artificially widened beach serving to nourish downdrift beaches by natural littoral currents or forces.

FETCH - Area where waves are generated by wind which has steady direction and speed. Sometimes called FETCH LENGTH.

FETCH LENGTH - Horizontal direction (in the wind direction) over which a wind generates waves. In sheltered waters, often the maximum distance that wind can blow across water.

FILTER CLOTH - Synthetic textile with openings for water to escape, but which prevents passage of soil particles.

FLOOD CURRENT - The tidal current toward shore or up a tidal stream, usually associated with the increase in the height of the tide.

FLOOD TIDE - The period of tide between low water and the succeeding high water; a rising tide.

FOREDUNE - The front dune immediately behind the backshore.

FORESHORE - The part of the shore lying between the crest of the seaward berm (or upper limit of wave wash at high tide) and the ordinary low water mark, that is ordinarily traversed by the uprush and backrush of the waves as the tides rise and fall.

FREEBOARD - The additional height of a structure above design high water level to prevent overflow. Also, at a given time, the vertical distance between the water level and the top of the structure. On a ship, the distance from the water-line to main deck or gunwale.

FUNCTIONAL LIFE - The period of time the structure performs as intended. Performance can be expressed in terms of benefits obtained versus the cost of maintenance.

GRADIENT (GRADE) - See SLOPE. With reference to winds or currents, the rate of increase or decrease in speed, usually in the vertical; or the curve that represents this rate.

GRAVEL - Small, rounded granules of rock with individual diameters ranging from 3.0 to 0.18 inches. Gravels are intermediate between SAND and COBBLES.

GROIN - Shore protection structure built perpendicular to shore to trap sediment and retard shore erosion.

GROIN FIELD - Series of groins acting together to protect a section of beach. Also called a groin system.

GROUND WATER - Subsurface water occupying the zone of saturation. In a strict sense, the term is applied only to water below the WATER TABLE.

HEADLAND (HEAD) - A high steep-faced promontory extending into the sea.

HIGHER HIGH WATER (HHW) - The higher of the two high waters of any tidal day. The single high water occurring daily during periods when the tide is diurnal is considered to be a higher high water.

HIGHER LOW WATER (HLW) - The higher of two low waters of any tidal day.

HIGH TIDE, HIGH WATER (HW) - The maximum elevation reached by each rising tide. See TIDE.

HIGH WATER LINE - In strictness, the intersection of the plane of mean high water with the shore. The shoreline delineated on the nautical charts of the U.S. Coast and Geodetic Survey is an approximation of the high water line. For specific occurrences, the highest elevation on the shore reached during a storm or rising tide, including meteorological effects.

IMPERMEABLE - Not permitting passage of water.

IMPERMEABLE GROIN - A groin through which sand cannot pass.

INLET - (1) A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water. (2) An arm of the sea (or other body of water), that is long compared to its width, and may extend a considerable distance inland.

INTERTIDAL ZONE - Land area alternately inundated and uncovered in tides. Usually considered to extend from MEAN LOW WATER to MEAN HIGH WATER.

JETTY - (1) (U.S. usage) On open seacoasts, a structure extending into a body of water, and designed to prevent shoaling of a channel by littoral materials, and to direct and confine the stream or tidal flow. Jetties are built at the mouth of a river or tidal inlet to help deepen and stabilize a channel. (2) (British usage) Jetty is synonymous with "wharf" or "pier."

LITTORAL - Of or pertaining to a shore.

LITTORAL DRIFT - The sedimentary material moved in the littoral zone under the influence of waves and currents.

LITTORAL TRANSPORT - The movement of littoral drift in the littoral zone by waves and currents. Includes movement parallel (longshore transport) and perpendicualr (on-offshore transport) to the shore.

LITTORAL TRANSPORT RATE - Rate of transport of sedimentary material parallel to or perpendicular to the shore in the littoral zone. Usually expressed in cubic yards (meters) per year. Commonly used as synonymous with LONGSHORE TRANSPORT RATE.

LITTORAL ZONE - In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.

LONGSHORE - Parallel to and near the shoreline.

LONGSHORE TRANSPORT RATE - Rate of transport of sedimentary material parallel to the shore. Usually expressed in cubic yards (meters) per year. Commonly used as synonymous with LITTORAL TRANSPORT RATE.

LOWER HIGH WATER (LHW) - The lower of the two high waters of any tidal day.

LOWER LOW WATER (LLW) - The lower of the two low waters of any tidal day. The single low water occurring daily during periods when the tide is diurnal is considered to be a lower low water.

LOW TIDE (LOW WATER, LW) - The minimum elevation reached by each falling tide.

LOW WATER LINE - The intersection of any standard low tide datum plane with the shore.

MARSH - An area of soft, wet, or periodically inundated land, generally treeless and usually characterized by grasses and other low growth.

MARSH, SALT - A marsh periodically flooded by salt water.

MEAN HIGHER HIGH WATER (MHHW) - The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN HIGH WATER (MHW) - The average height of the high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.

MEAN LOWER LOW WATER (MLLW) - The average height of the lower low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. Frequently abbreviated to LOWER LOW WATER.

MEAN LOW WATER (MLW) - The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to elliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

MEAN LOW WATER SPRINGS - The average height of low waters occurring at the time of the spring tides. It is usually derived by taking a plane depressed below the half-tide level by an amount equal to one-half the spring range of tide, necessary corrections being applied to reduce the result to a mean value. This plane is used to a considerable extent for hydrographic work outside of the United States and is the plane of reference for the Pacific approaches to the Panama Canal. Frequently abbreviated to LOW WATER SPRINGS.

MEAN SEA LEVEL - The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. Not necessarily equal to MEAN TIDE LEVEL.

MEAN TIDE LEVEL - A plane mideway between MEAN HIGH WATER AND MEAN LOW WATER. Not necessarily equal to MEAN SEA LEVEL. Also called HALF-TIDE LEVEL.

MIXED TIDE - A tide in which there is a distinct difference in height between successive high and successive low waters. For mixed tides there are generally two high and two low waters each tidal day. Mixed tides may be described as intermediate between semidiurnal and diurnal tides.

MUD - A fluid-to-plastic mixture of finely divided particles of solid material and water.

NAUTICAL MILE - The length of a minute of arc, 1/21,600 of an average great circle of the earth. Generally one minute of latitude is considered equal to one nautical mile. The accepted United States value as of 1 July 1959 is 6,076.115 feet or 1,852 meters, approximately 1.15 times as long as the statute mile of 5,280 feet. Also geographical mile.

NEAP TIDE - A tide occurring near the time of quadrature of the moon with the sun. The neap tidal range is usually 10 to 30 percent less than the mean tidal range.

NEARSHORE (ZONE) - In beach terminology an indefinite zone extending seaward from the shoreline well beyond the breaker zone. It defines the area of NEARSHORE CURRENTS.

NEARSHORE CURRENT SYSTEM - The current system caused primarily by wave action in and near the breaker zone, and which consists of four parts: The shoreward mass transport of water; longshore currents; seaward return flow, including rip currents; and the longshore movement of the expanding heads of rip currents.

NEAT LINES - Lines on drawings which establish tolerances for construction.

NODAL ZONE - An area in which the predominant direction of the LONGSHORE TRANSPORT changes.

NOURISHMENT - The process of replenishing a beach. It may be brought about naturally, by longshore transport, or artificially by the deposition of dredged materials.

OFFSHORE - (1) In beach terminology, the comparatively flat zone of variable width, extending from the breaker zone to the seaward edge of the Continental Shelf. (2) A direction seaward from the shore.

OFFSHORE CURRENT - (1) Any current in the offshore zone. (2) Any current flowing away from shore.

OVERTOPPING - Passing of water over the top of a structure as a result of wave runup or surge action.

OVERWASH - That portion of the uprush that carries over the crest of a berm or of a structure.

PERCOLATION - The process by which water flows through the interstices of a sediment. Specifically, in wave phenomena, the process by which wave action forces water through the interstices of the bottom sediment. Tends to reduce wave heights.

PERMEABLE- Having openings large enough to permit free passage of appreciable quantities of sand or water.

PERMEABLE GROIN - A groin with openings large enough to permit passage of appreciable quantities of littoral drift.

PIER - A structure, usually of open construction, extending out into the water from the shore, to serve as a landing place, a recreational facility, etc., rather than to afford coastal protection. In the Great Lakes, a term sometimes improperly applied to jetties.

PILE - A long, heavy timber of section of concrete or metal to be driven or jetted into the earth or seabed to serve as a support or protection.

PILE, SHEET - A pile with a generally slender flat cross section to be driven into the ground or seabed and meshed or interlocked with like members to form a diaphragm, wall, or bulkhead.

PILING - A group of piles.

PLANFORM - The outline or shape of a body of water as determined by the stillwater line.

PROFILE, BEACH - Intersection of the ground surface with a vertical plane that may extend from the top of the dune line to the seaward limit of sand movement.

RECESSION (of a beach) - (1) A continuing landward movement of the shoreline. (2) A net landward movement of the shoreline over a specified time.

REFERENCE STATION - A place for which tidal constants have previously been determined and which is used as a standard for the comparison of simultaneous observations at a second station; also a station for which independent daily predictions are given in the tide or current tables from which corresponding predictions are obtained for other stations by means of differences or factors.

REFLECTED WAVE - That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

REFRACTION (OF WATER WAVES) - (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed. The part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) The bending of wave crests by currents.

REVETMENT - A facing of stone, concrete, etc., built to protect a scarp, enbankment, or shore structure against erosion by wave action or currents.

RIPRAP - A layer, facing, or protective mound of stones randomly placed to prevent erosion, scour, or sloughing of a structure or embankment; also the stone so used.

RUBBLE - (1) Loose angular waterworn stones along a beach. (2) Rough, irregular fragments of broken rock.

RUBBLE-MOUND STRUCTURE - A mound of random-shaped and random-placed stones protected with a cover layer of selected stones or specially shaped concrete armor units. (Armor units in primary cover layer may be placed in orderly manner or dumped at random.)

SAND - Generally, coarse-grained soils having particle diameters between 0.18 and approximately 0.003 inches. Sands are intermediate between SILT and GRAVELS.

SAND FILLET - Accretion trapped by a groin or other protrusion in the littoral zone.

SCARP, BEACH - An almost vertical slope along the beach caused by erosion by wave action. It may vary in height from a few inches to several feet, depending on wave action and the nature and composition of the beach.

SCOUR - Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

SEAWALL - A structure separating land and water areas, primarily designed to prevent erosion and other damage due to wave action. See also BULKHEAD.

SEMIDIURNAL TIDE - A tide with two high and two low waters in a tidal day with comparatively little diurnal inequality.

SHALLOW WATER - (1) Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water. (2) More strictly, in hydrodynamics with regard to progressive gravity waves, water in which the depth in less than 1/25 the wavelength. Also called VERY SHALLOW WATER.

SHINGLE - (1) Loosely and commonly, any beach material coarser than ordinary gravel, especially any having flat or flattish pebbles. (2) Strictly and accurately, beach material of smooth, well-rounded pebbles that are roughly the same size. The spaces between pebbles are not filled with finer materials. Shingle often gives out a musical sound when stepped on.

SHOAL - (noun) Rise of the sea bottom from an accumulation of sand or other sediments. (verb) - (1) to <u>become</u> shallow gradually. (2) To <u>cause</u> to become shallow. (3) to <u>proceed</u> from a greater to a lesser depth of water.

SHORE - Narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a beach.

SHOREFACE - The narrow zone seaward from the low tide SHORELINE covered by water over which the beach sands and gravels actively oscillate with changing wave conditions.

SHORELINE - The intersection of a specified plane of water with the shore or beach. (e.g., the highwater shoreline would be the intersection of the plane of mean high water with the shore or beach.) The line delineating the shoreline on U.S. Coast and Geodetic Survey nautical charts and surveys approximates the mean high water line.

SILT - Generally refers to fine-grained soils having particle diameters between 0.003 and 0.00015 inches. Imtermediate between CLAY and SAND.

SLOPE - Degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25 or 1 on 25, indicating 1 unit vertical rise in 25 units of horizontal distance; or in degrees from horizontal.

SPECIFICATIONS - Detailed description of particulars, such as size of stone, quality of materials, contractor performance, terms, and quality control.

STILLWATER LEVEL - The elevation that the surface of the water would assume if all wave action were absent.

STORM SURGE - A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress.

SURF - The wave activity in the area between the shoreline and the outermost limit of breakers.

SURF ZONE - The area between the outermost breaker and the limit of wave uprush.

SURGE - (1) The name applied to wave motion with a period intermediate between that of the ordinary wind wave and that of the tide, say from 1/2 to 60 minutes. It is of low height; usually less than 0.3 foot. See also SEICHE. (2) In fluid flow, long interval variations in velocity and pressure, not necessarily periodic, perhaps even transient in nature.

SWASH - The rush of water up onto the beach face following the breaking of a wave.

TIDAL PERIOD - The interval of time between two consecutive like phases of the tide.

TIDAL RANGE - Difference in height between consecutive high and low (or higher and lower low) waters. The mean range is the difference in height between mean high water and mean low water. The diurnal range is the difference in height between mean higher high water and mean lower low water. For diurnal tides, the mean and diurnal range are identical. For semidiurnal and mixed tides, the spring range is the difference in height between the high and low waters during the time of spring tides.

TIDE - The periodic rising and falling of the water that results from gravitational attraction of the moon and sun and other astronomical bodies acting upon the rotating earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable tc designate the latter as TIDAL CURRENT, reserving the name TIDE for the vertical movement.

TIDE, DIURNAL - A tide with one high water and one low water in a tidal day.

TIDE STATION - A place at which tide observations are being taken. It is called a primary tide station when continuous observations are to be taken over a number of years to obtain basic tidal data for the locality. A secondary tide station is one operated over a short period of time to obtain data for a specific purpose.

TIE ROD - Steel rod used to tie back the top of a bulkhead or seawall.

WAVE PERIOD - The time for a wave crest to traverse a distance equal to one wavelength. The time for two successive wave crests to pass a fixed point.

WAVE, REFLECTED - That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

WAVE SPECTRUM - In ocean wave studies, a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency. The spectrum may be based on observations or theoretical considerations. Several forms of graphical display are widely used.

WAVE STEEPNESS - The ratio of the wave height to the wavelength.

WAVE TRAIN - A series of waves from the same direction.

WAVE TROUGH - Lowest part of a wave form between successive crests. Also, that part of a wave below the stillwater level.

WEEP HOLE - Hole through a solid revetment, bulkhead, or seawall for relieving pore pressure.

WEIR JETTY - An updrift jetty with a low section or weir over which littoral drift moves into a predredged deposition basin which is dredged periodically.

WHITECAP - On the crest of a wave, the white froth caused by wind.

WIND SETUP - The vertical rise in the stillwater level on the leeward side of a body of water caused by wind stresses on the surface of the water.

WINDWARD - The direction from which the wind is blowing.

WIND WAVES - (1) Waves being formed and built up by the wind.
(2) Loosely, any wave generated by wind.